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FINAL ENVIRONMENTAL PLANNING TECHNICAL REPORT



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WATER RESOURCES

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January 1984

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Air Force
Environmental Planning Division
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WATER RESOURCES

January 1984

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PREFACE

The President has directed that the Air Force deploy the Peacekeeper missile system at a location near F.E. Warren Air Force Base (hereafter F.E. Warren AFB), close to Cheyenne, Wyoming. The Peacekeeper system (formerly known as the M-X system) is an advanced, land-based intercontinental ballistic missile. The plan calls for the replacement of 100 existing Minuteman III missiles with 100 Peacekeeper missiles. Existing missile silos will be used, and there will be very little structural modification needed. Missile replacement will occur within the two squadrons (of 50 missiles each) located nearest F.E. Warren AFB, the 319th and 400th Strategic Missile Squadrons. Peacekeeper deployment will occur between 1984 and 1989.

An environmental impact statement (EIS) was prepared for the Proposed Action as outlined above. Information contained in the EIS is based upon environmental information and analysis developed and reported in a series of 13 final environmental planning technical reports (EPTRs). This volume is one of those reports. The 13 resource areas are:

- Socioeconomics (employment demand, housing, public finance, construction resources, and social well-being);
- Public Services and Facilities;
- Utilities;
- Energy Resources;
- Transportation;
- Land Use (land use, recreation, and visual resources);
- Cultural and Paleontological Resources;
- Water Resources;
- Biological Resources;
- Geologic Resources;
- Noise;
- Air Quality;
- Jurisdictional.

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1.0

INTRODUCTION

1.0 INTRODUCTION

This final environmental planning technical report (EPTR) is a companion document to the water resources section of the final environmental impact statement (FEIS) for the Peacekeeper in Minuteman Silos project. It provides data, methodologies, and analyses which supplement and extend those presented in the FEIS.

This final EPTR consists of six major sections. Section 1.0 provides an overview of the Peacekeeper in Minuteman Silos project and a description of water resources and their elements.

Section 2.0 presents a detailed description of the environment potentially affected by the project. It includes a capsule description of the environmental setting (Section 2.1) and project requirements (Section 2.2). Section 2.3 defines the Region of Influence and Area of Concentrated Study for the resource. Section 2.4 (Derivation of Data Base) follows with a discussion of the literature sources, group and agency contacts, and primary data which provide the data base for the report. Section 2.5 describes analytic methods used to determine existing environmental conditions in the Region of Influence. Detailed analyses of the existing environment, broken down by constituent elements of the resource, follow in Section 2.6.

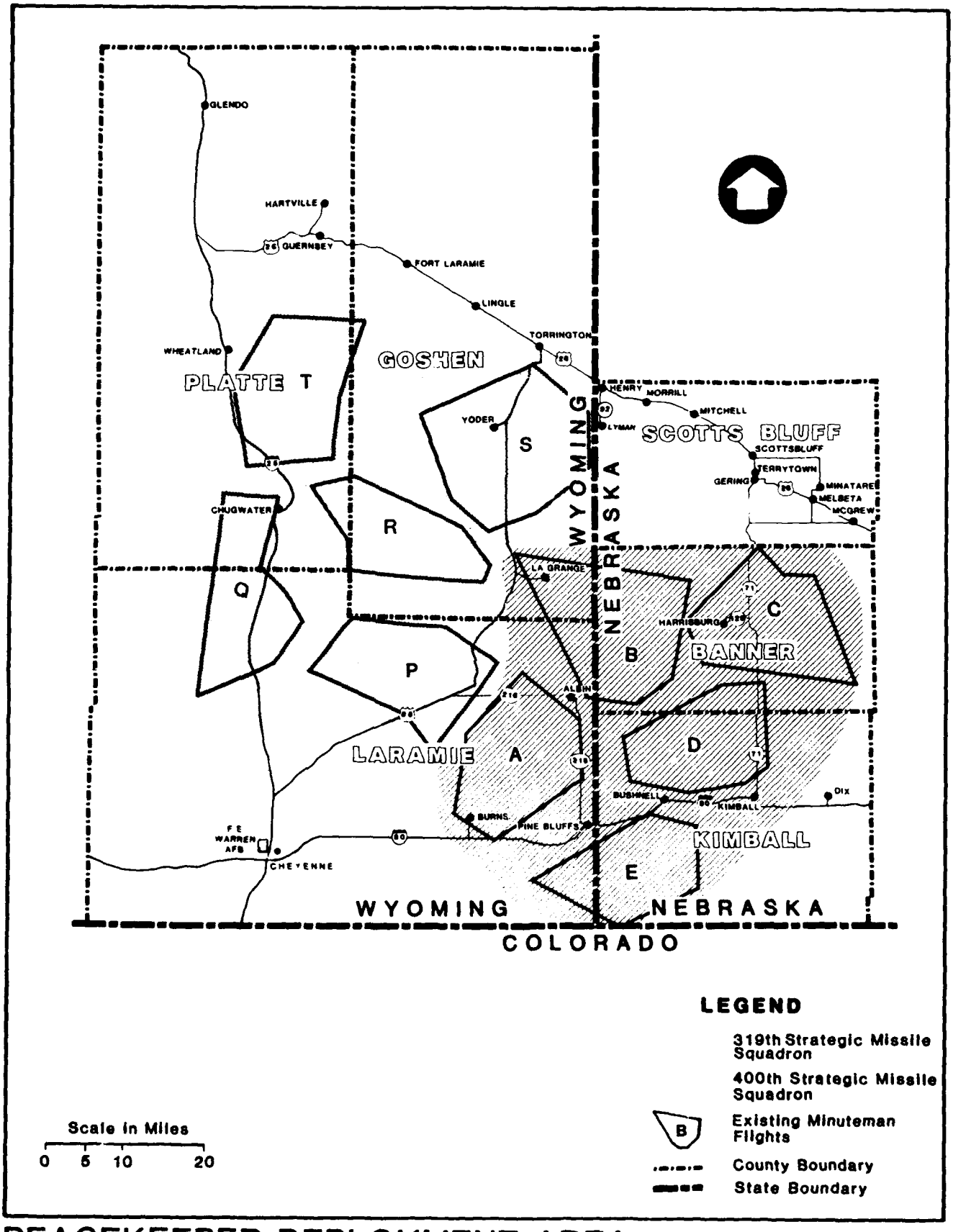
Section 3.0 describes environmental consequences of the Proposed Action and its project element alternatives, the No Action Alternative, mitigation measures, and unavoidable impacts. It contains detailed definitions of each potential level of impact (negligible, low, moderate, and high) for both short-term and long-term impacts. Beneficial effects are also discussed. Definitions of significance are also included. Methods used for analyzing future baseline and project impacts are described, as are assumptions and assumed mitigations. Additional mitigation measures to reduce project impacts are also described.

Sections 4.0 (Glossary), 5.0 (References), and 6.0 (List of Preparers) conclude the EPTR.

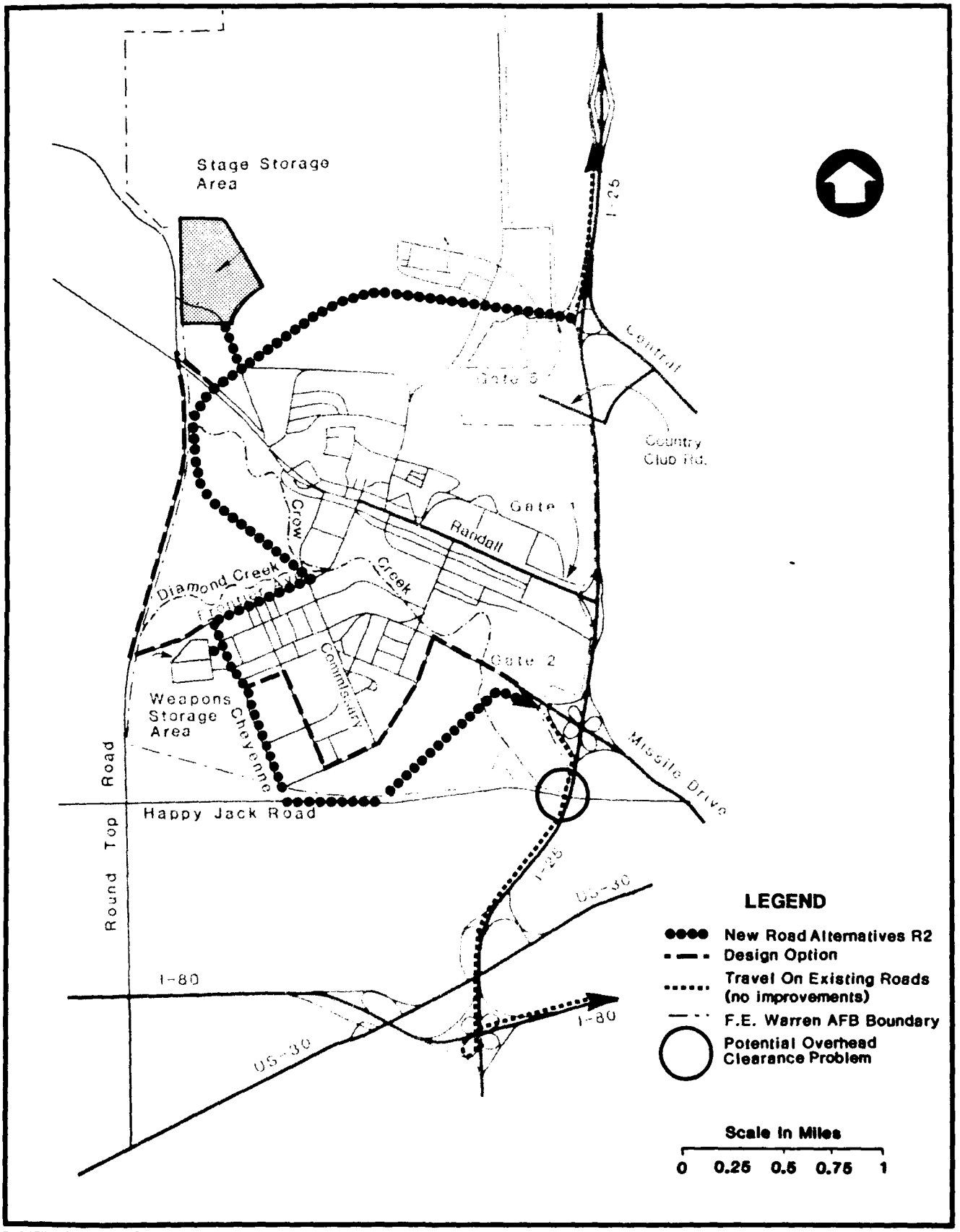
1.1 Peacekeeper in Minuteman Silos

The Peacekeeper system, which the Air Force plans to deploy within the 90th Strategic Missile Wing at F.E. Warren Air Force Base (AFB), Wyoming, is an advanced land-based intercontinental ballistic missile system designed to improve the nation's strategic deterrent force. Deployment of the Peacekeeper calls for replacement of 100 existing Minuteman III missiles with 100 Peacekeeper missiles. Missile replacement will occur in the 319th and 400th Strategic Missile Squadrons, located nearest F.E. Warren AFB (Figure 1.1-1). The Deployment Area covers parts of southeastern Wyoming and the southwestern Nebraska Panhandle.

Construction at F.E. Warren AFB will occur between 1984 and 1986. Fourteen new buildings will be constructed, and modifications or additions will be made to 11 existing buildings. Approximately 400,000 square feet of floor space will be built or modified. A new road configuration, to be selected from three alternatives, is proposed to link Peacekeeper facilities onbase and to provide improved access to or from the base (Figures 1.1-2, 1.1-3, and 1.1-4).

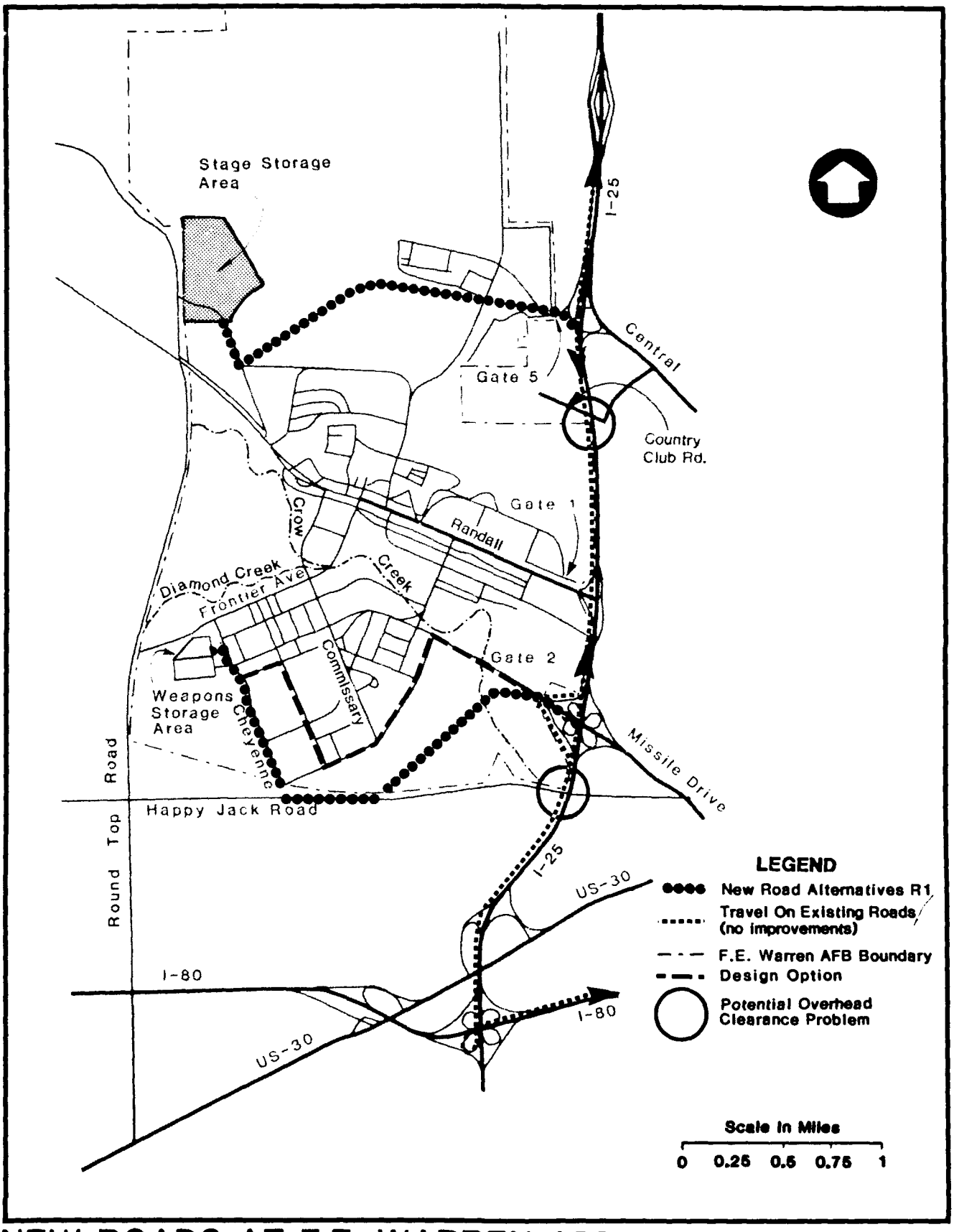


PEACEKEEPER DEPLOYMENT AREA FIGURE NO. 1.1-1



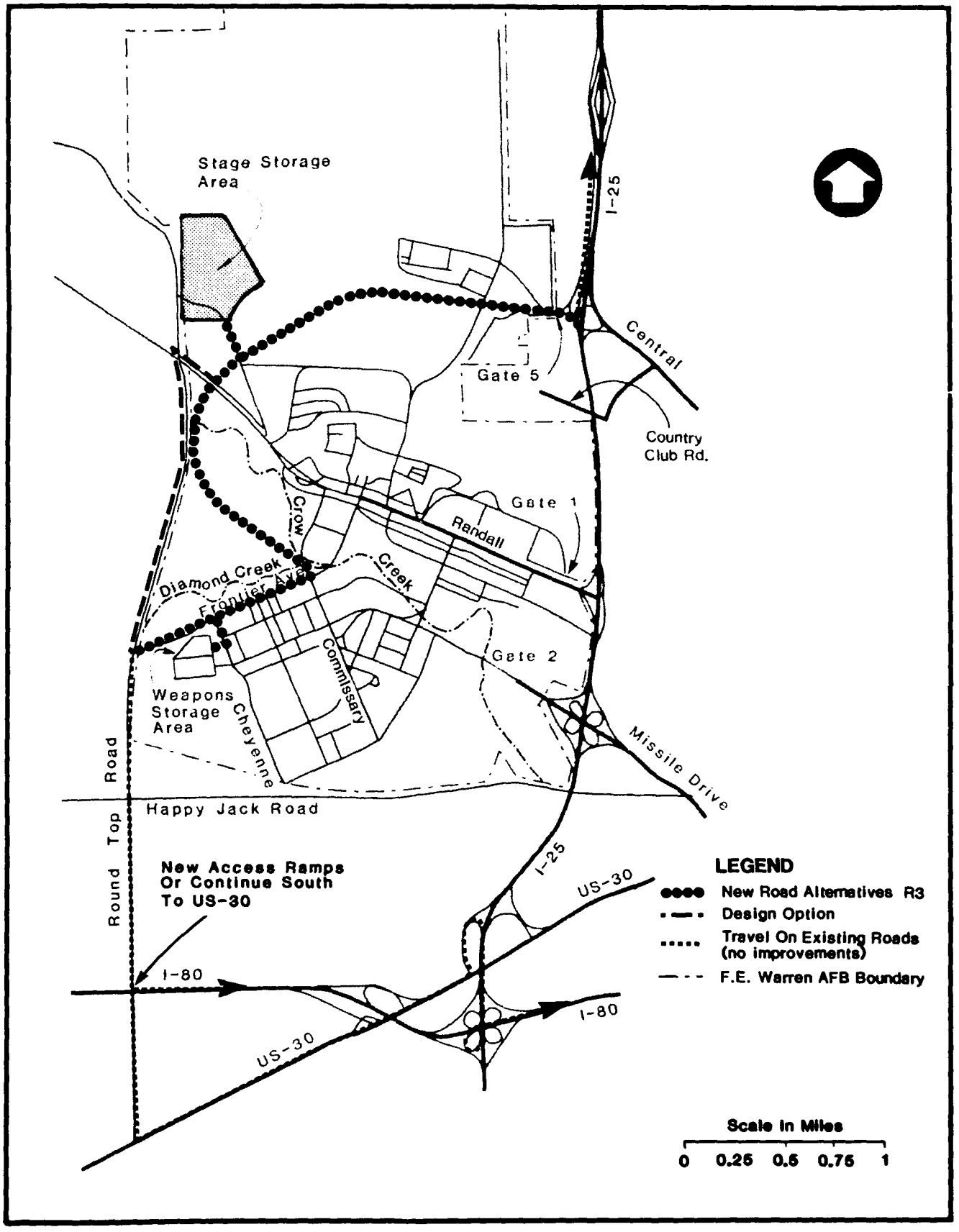
NEW ROADS AT F.E. WARREN AFB: PROPOSED ACTION R2

FIGURE NO. 1.1-2



NEW ROADS AT F.E. WARREN AFB: ALTERNATIVE R1

FIGURE NO. 1.1-3



NEW ROADS AT F.E. WARREN AFB: ALTERNATIVE: R3

FIGURE NO. 1.1-4

Work in the Deployment Area will take place between 1985 and 1989. Many of the access roads to the Launch Facilities will be upgraded. Bridge clearance problems will be corrected, and some culverts and bridges may need to be upgraded. Below-ground modifications will be related to removal of Minuteman support hardware, insertion of a protective canister to enclose the Peacekeeper, and installation of communications systems and support equipment.

A total of 11 alternatives have been chosen as candidate routes for communication connectivity between Squadrons 319 and 400 (Figure 1.1-5). Five routes will be selected for installation. Total buried cable length will range from approximately 82 to 110 miles, depending upon final route selections.

Under the Proposed Action two dispatch stations would be established, one each in the northern and eastern portions of the Deployment Area. Although actual locations have not been selected, Chugwater, Wyoming and Kimball, Nebraska are representative locations analyzed in the Final Environmental Impact Statement and in this EPTR. Dispatch stations would be not more than 5 acres in size and would be used for the temporary open storage of equipment and material. One or more buildings would also be present at each site for contractor use as office space. All dispatch stations would be removed prior to project completion. In addition to the Proposed Action, two alternatives are considered in this environmental impact assessment:

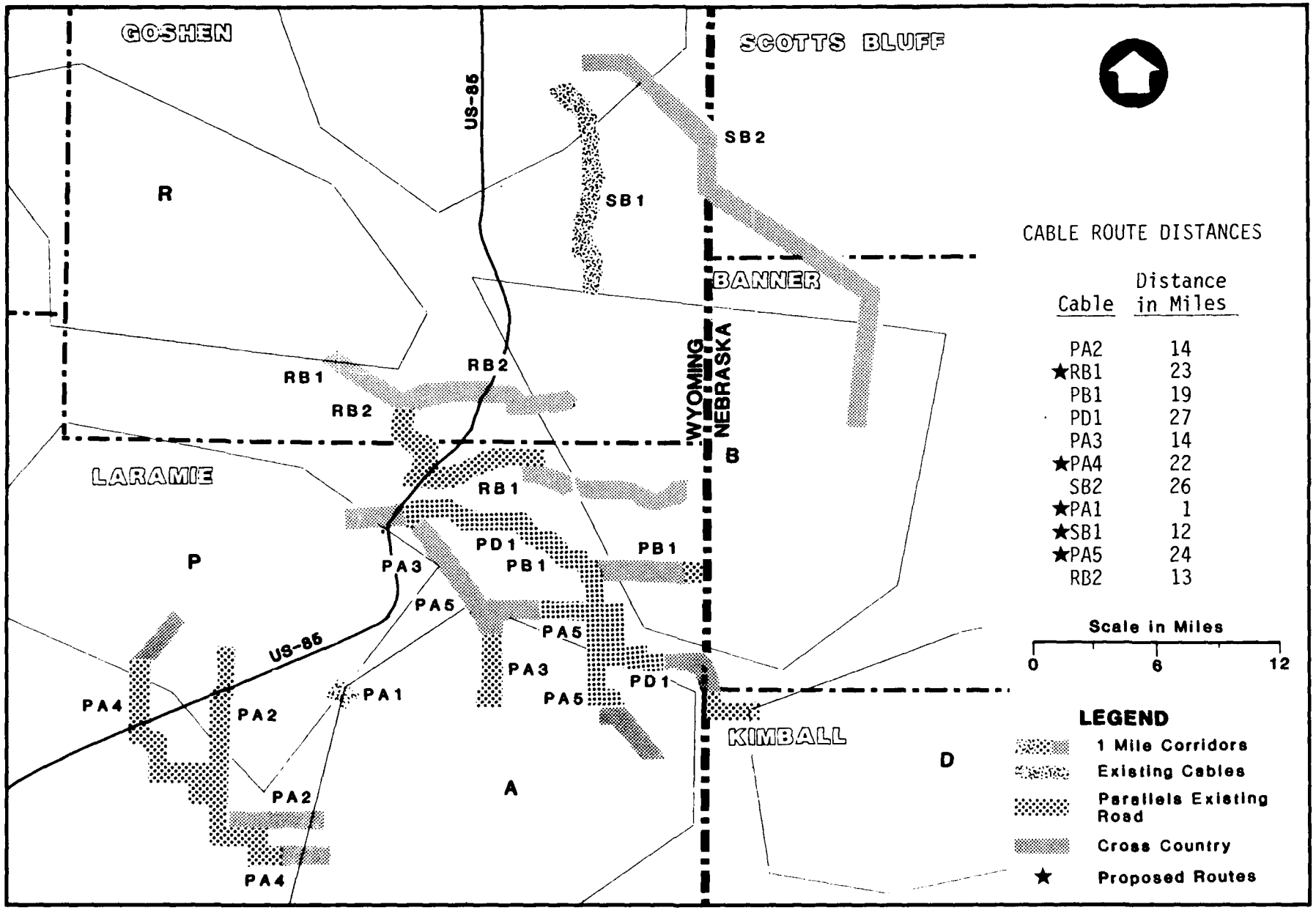
1) One dispatch station only, in the eastern part of the Deployment Area; or

2) No dispatch stations.

Two options have been identified for resurfacing Deployment Area roads. Surfacing Option A involves gravel upgrades of 252 miles of existing gravel roads and the paving or repaving of 390 additional miles of gravel and asphalt roads. Surfacing Option B involves the paving or repaving of all 642 miles of gravel and asphalt roads listed in Surfacing Option A.

Direct manpower for construction, assembly and checkout, and operation of the system will peak during 1986 when an average of nearly 1,600 persons will be required. In 1991, following deployment, the remaining increased operational workforce at F.E. Warren AFB will consist of about 475 persons. Table 1.1-1 presents the average annual workforce, based on quarterly estimates for each year of construction.

Table 1.1-2 shows the average number of jobs including those which are considered to be filled by available labor; as well as those filled by weekly commuters and inmigrants, on an annual average basis. In general, locally available labor will fill all the road and construction jobs.



ALTERNATIVE CABLE ROUTES

FIGURE 1.1-5

Table 1.1-1

PROJECT AVERAGE MANPOWER REQUIREMENTS BY YEAR[1]

	1984	1985	1986	1987	1988	1989	1990	1991
Deployment Area								
Construction	5	40	60	60	40	0	0	0
Assembly and Checkout	0	15	210	285	265	265	10	0
Operations	0	0	0	0	0	0	0	0
Defense Access Road	0	275	315	150	0	0	0	0
Subtotal	5	330	585	495	305	265	10	0
Operating Base								
Construction	100	630	70	0	0	0	0	0
Assembly and Checkout	40	130	525	555	515	510	22	0
Operations	0	130	415	490	500	500	475	475
Subtotal	140	890	1,010	1,045	1,015	1,010	497	475
TOTAL:	145	1,220	1,595	1,540	1,320	1,275	507	475

Note: [1] Estimates based on average quarterly employment.

Table 1.1-2

TOTAL JOBS, LOCAL AND REGIONAL HIRES, AND INMIGRATION FOR THE EMPLOYMENT DEMAND REGION OF INFLUENCE

	1984	1985	1986	1987	1988	1989	1990	1991 and beyond
1) Total (Direct/ Indirect) Additional Jobs	250	2,400	2,675	2,550	2,025	1,825	650	590
2) Average Annual Local Hires	157	1,750	1,525	1,350	1,100	815	225	230
3) Average Annual Weekly Commuters	25	225	175	100	25	10	0	0
4) Average Annual Inmigrant Workers	75	425	950	1,100	925	1,000	425	360
5) Unsuccessful Job-Seekers	30	185	180	150	165	110	70	0
6) Inmigrant[1] Population	275	1,475	2,875	3,200	3,025	2,875	1,200	925

Note: [1] Includes inmigrants, workers, and unsuccessful job-seekers.

As a result of the purchase of materials in the project area and the local expenditures of project employees, additional jobs will be created in the region. These jobs are estimated to number as follows:

Year:	1984	1985	1986	1987	1988	1989	1990	1991 & on
Indirect Jobs:	105	1,180	1,080	1,010	705	550	143	115

Estimated materials and costs for the project, based on total project budgetary considerations, are shown by Standard Industrial Classification in Table 1.1-3.

A number of construction and support materials will be obtained from sources within the project area. Among the materials exerting a major influence on assessment of project impacts are aggregate (4.6 million tons), water (516 acre-feet), fuel (7.6 million gallons), and electricity (3.8 million kWh). In the case of water supply for construction, the Air Force will identify and, if necessary, obtain permits for the water or purchase existing water rights.

1.2 Description of the Resources

The water resources system consists of groundwater and surface water, man's use of those resources, and man's control of that use as it affects water quantity and quality.

This report describes the existing conditions and anticipated future trends for water resources in the area potentially affected by construction and long-term operation of the Peacekeeper in Minuteman Silos project at F.E. Warren AFB in Wyoming and in Minuteman Wing V in Wyoming and Nebraska.

Water will be required for project construction, operation, and to satisfy demands induced by inmigrating workers, dependents, and related induced activities in parts of Wyoming and Nebraska. Construction of facilities may alter natural drainageways and impact stormwater runoff with resulting erosion and sedimentation. Increased water use will result in increased wastewater flows which can impact the natural water resource system as it is returned to the environment.

This report assesses effects on water resources of the Proposed Action and of the No Action Alternative. The level of impact resulting from these effects is estimated, as is the potential significance of those impacts. Methods of mitigating potentially significant impacts are also presented.

The water resources system analysis has been divided into four elements: water use and demand; constraints on water use; surface water hydrology and quality; and groundwater hydrology and quality.

Table 1.1-3

ESTIMATED MATERIAL REQUIREMENTS
BY STANDARD INDUSTRIAL CLASSIFICATION

Industrial Classification		Estimated 1982 Dollars (1,000s)
Fabricated Structural Metal		$22,999
Unclassified Professional Services and Products		14,358
Cement and Concrete Products		10,862
General Wholesale Trade		8,890
Structural Metal Products[1]	11,983	
Millwork, Plywood, and Wood Products[1]		3,941
Copper, Copper Products		3,902
Electrical Lighting and Wiring		3,871
Stone and Clay Mining and Quarrying		39,728
Stone and Clay Products[1]		2,955
Basic Steel Products		1,233
Heating and Air Conditioning Apparatus		1,525
Plumbing and Plumbing Fixtures		938
Petroleum Refining and Products		5,148
Material Handling Equipment		1,970
Sawmills and Planing Mills		1,478
Paints and Allied Products		1,478
Plastic Products[1]		1,478
Furniture and Fixtures		986
Structural Clay Products		986
General Hardware		986
Scientific Instruments		986
Rail Transport		986
Real Estate		986
Construction, Mining, and Oilfield Machinery		749
TOTAL:		$145,402

Note: [1] Not included in other Industrial Classifications.

1.2.1 Water Use and Demand

The water use and demand element addresses municipal water use in the Cheyenne Urban Area, other municipal water use, and nonmunicipal use for residential, commercial, industrial, agricultural irrigation, and livestock water uses. The supply sources used to meet water demands are also included.

1.2.2 Constraints on Water Use

The constraints on water use element addresses water rights and other controls on water use and laws and regulations that protect water quality.

1.2.3 Surface Water Hydrology and Quality

The surface water hydrology and quality element addresses streams, lakes, ponds, and reservoirs, their flow characteristics, and water quality conditions.

1.2.4 Groundwater Hydrology and Quality

The groundwater hydrology and quality element addresses aquifers and recharge to and discharge from those aquifers as well as water quality conditions in the aquifers.

2.0

AFFECTED ENVIRONMENT

2.0 AFFECTED ENVIRONMENT

2.1 General

The proposed project area is located in the Platte River Basin which is part of the larger Missouri River Basin. Both surface water and groundwater resources are limited in the project area due to hydrologic conditions and the extent of present water use. Because of these conditions, increased use of water for water supply or other changes in the hydrologic system that can impact water quantity or quality are of concern.

The Crow Creek watershed in and around Cheyenne, Wyoming, is typical of many of the watersheds in the project area in that it is drained by a small stream with widely varying but seasonally small flow, has been developed to some extent to satisfy present water demands, and is underlain by groundwater aquifers that supplement surface water supplies. Many of the project activities could influence water resources in this watershed. Other project activities could influence hydrologic conditions in other watersheds in the project area during project construction. Water resource conditions are not expected to be markedly different as a result of the operational period of the project from those that now exist.

This section initially presents information on direct and indirect project water requirements. A Region of Influence (ROI) and Areas of Concentrated Study (ACSs) for each water resource element are then defined based on likely effects of this water use or other project activities. The ROI outlines an area where data were collected and analytic methods were applied to describe existing environmental conditions. The section includes a summary of methods used to collect and analyze the data. Existing environmental conditions that could be influenced by the project are then described. Effects of the Proposed Action or No Action Alternative are assessed in the following section.

2.2 Project Requirements

Overall project requirements are outlined in Section 1.1. Requirements specific to water resources are as follows.

2.2.1 Direct Project Requirements

Water will be required for construction at F.E. Warren AFB; in and around the 319th and 400th Squadrons Deployment Area (DA) in conjunction with work at Launch Control Facilities (LCFs) and Launch Facilities (LFs); and in portions of Banner, Scotts Bluff, and Kimball counties, Nebraska, and Laramie, Goshen, and Platte counties, Wyoming, in conjunction with the Defense Access Road (DAR) upgrade program and communication links (cable routes). Long-term operations requirements will occur at F.E. Warren AFB and at LCFs. Future LCF water use is not expected to exceed present LCF water use.

Estimates of construction water requirements by year and by location are provided in Table 2.2-1. The major uncertainty in the estimate is the DAR road program, the extent of which requires additional engineering investigation and discussion with local officials. The present estimate is based on

Table 2.2-1

CONSTRUCTION WATER USE
(acre-ft)

Use	1983	1984	1985	1986	1987	1988	1989	Total	Place of Use DA	Place of Use F.E. Warren AFB
Compaction, Roads[1]	0	1.55	3.20	0.75	0.75	0.75	0	7.00	3.00	4.00
Road Upgrade, DAk/AF Roads	0	4.10	75.70	83.90	40.90	0	0	204.6	204.6	0
Compaction, Parking Lot	0	0.20	0	0	0	0	0	0.20	0	0.20
Concrete Building/Pads	0	0.45	2.80	1.00	0.75	0.50	0	5.50	2.5	3.0
Aggregate Building/Pads	0	2.10	13.70	4.85	3.55	2.35	0	26.55	11.90	14.65
Compaction Building/Pads	0	0.45	3.10	1.10	0.80	0.55	0	6.00	2.70	3.30
Revegetation, Building	0	1.60	6.15	0.80	0	0	0	8.55	0	8.55
Subtotal:	0	10.45	104.65	92.40	46.75	4.15	0	258.4	224.70	33.70
Contingency,[2] 100%	0	10.45	104.65	92.40	46.75	4.15	0	258.4	224.70	33.70
TOTAL:	0	20.90	209.30	184.80	93.50	8.30	0	516.80	449.4	67.40
Place of Use										
F.E. Warren AFB	0	12.70	49.30	5.40	0	0	0	67.40	-	-
DA	0	8.20	160.00	179.40	93.50	8.30	0	449.40	-	-

Notes: 1 Roads at LF and at F.E. Warren AFB.
2 Estimate for culvert, bridgework, cables installation, and miscellaneous subgrade improvements and dust control.

the worst-case upgrade involving resurface work on about 340 miles of county and Air Force gravel roads and resurface work on about 305 miles of presently paved roads. A 100-percent contingency was then added to obtain the 516 acre-feet (acre-ft) estimate for all project construction work for the period of 1984 to 1989 when construction would be complete.

The project will change or disturb some land at each LF, at F.E. Warren AFB, at dispatch stations, and along roads and cable routes. This is estimated at about 0.5 acre per LF, an average of about 3.3 acres per mile of road, and about 130 acres at F.E. Warren AFB. These values are used for the purpose of storm drainage and erosion evaluation. It is also assumed that the installation of new communication facilities will disturb about 2.4 acres per mile.

2.2.2 Indirect Project Requirements

Indirect or induced water demands will occur as a result of inmigration of population. The socioeconomics task group has evaluated the number and location of this population change. Population values were converted to water demand values as follows: Cheyenne Urban Area - 180 gallons per capita per day (gpcd), other municipalities - 250 gpcd, and weekly commuters and transients in all areas - 100 gpcd.

Development of these factors is described in later sections of this report. The application of these factors to anticipated inmigration population by location to develop induced water demands is summarized in Table 2.2-2.

The total induced demand for all areas for 1984 to 1990 is 3,418 acre-ft with 2,732 in the Cheyenne Urban Area. A continued operational period induced demand of 187 acre-ft/yr is forecast for the Cheyenne Urban Area. In addition, operation requirements at F.E. Warren AFB, based on 150 gallons per day per thousand square feet of new buildings, will total 7 acre-ft in 1985 and about 63 acre-feet per year (acre-ft/yr) in 1986 and later years.

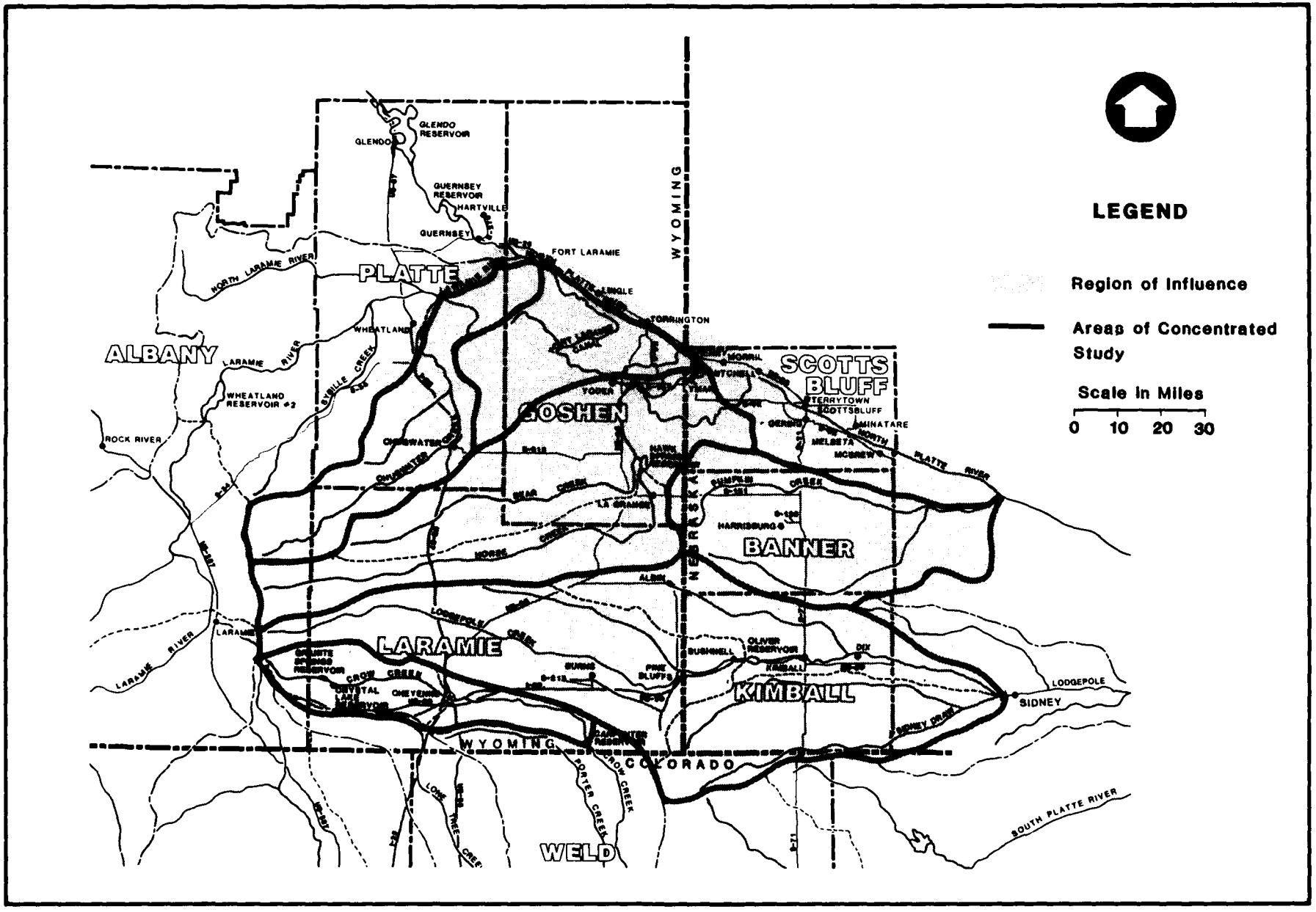
The indirect population will induce changes in urban characteristics of land use in Cheyenne which are described in later sections of this report.

2.3 Region of Influence

The ROI is defined as the area where direct and indirect project requirements can cause a measurable change in existing environmental conditions regardless of the magnitude or significance of that change.

The ROI includes locations where additional water will have to be provided to serve the direct and indirect project-induced population or demands arising from project construction requirements. The socioeconomics task group's population allocation model generates a population ROI used to determine which watersheds may potentially be affected. Watershed boundaries are superimposed over the direct project sites and the population ROI to form the outermost water resources ROI. That area is shown on Figure 2.3-1.

The water resource analysis was conducted for several ACSs within the overall water resources ROI. One of the ACSs was the Crow Creek watershed in Wyoming, since most anticipated direct water resource impacts would occur in this watershed upstream of the Wyoming-Colorado border, primarily around F.E. Warren AFB and Cheyenne.



REGION OF INFLUENCE FOR WATER RESOURCES

FIGURE NO. 2.3-1

Table 2.2-2

INDUCED WATER DEMANDS
(acre-ft/yr)

	1984	1985	1986	1987	1988	1989	1990	1991
Cheyenne Urban Area								
Cheyenne Inmigrants	50	243	439	505	468	449	228	187
Weekly Commuters	3	8	14	3	0	0	0	0
F.E. Warren AFB								
Operation		7	63	63	63	63	63	63
TOTAL:	53	258	516	571	531	512	291	250

Urban Area	Other Municipalities (acre-ft) Peak Year	Total (1984-1989)
Scottsbluff	61	142
Gering	31	72
Kimball	80	110
Pine Bluffs	42	45
Torrington	59	59
Wheatland	121	211
Chugwater	15	44
Albin	1	3
TOTAL:	165[a]	686

Note: a The actual projected use ranges from 148 to 165 acre-ft/yr for 1986 to 1989.

Water is imported into this ACS and flows out of it. This ACS for the Proposed Action does not extend to those areas from which water is imported, since the environmental impact, of bringing water to the area or increasing such imports are independent of the proposed project and have been previously addressed in a recent environmental impact statement (USFS 1981).

Other ACSs in the ROI include selected watersheds in the remainder of the initially defined ROI. Direct project construction-related water resource impacts in the DA and potentially significant population-induced impacts on municipal water supply and wastewater disposal could occur in municipalities in the selected watersheds.

Most anticipated project-related effects are not expected to be large in the other ACSs. These ACSs are defined below for each water resource element and include locations where it is probable that significant impacts could occur. This includes areas around LFs, cables, and DAR roads located inside the DA and in municipalities where population is allocated.

The ACSs within the overall water resources ROI are further defined in the following paragraphs. The basis for selecting the ACS is further discussed in Section 3.1.

2.3.1 Water Use and Demand

The ACS for water use and demand includes the Crow Creek watershed and those portions of counties in the DA (Banner, Kimball, Scotts Bluff, Laramie, Platte, and Goshen) in the overall water resources ROI.

2.3.2 Constraints on Water Use

The ACS for constraints on water use includes Wyoming and Nebraska requirements and constraints relative to water use and water quality that apply in the overall water resources ROI.

2.3.3 Surface Water Hydrology and Quality

The ACS for surface water hydrology and quality includes the Crow Creek watershed upstream of the Carpenter area and the Lodgepole, Horse, Pumpkin, and Chugwater Creek watersheds in the overall water resources ROI.

2.3.4 Groundwater Hydrology and Quality

The ACS for groundwater hydrology and quality includes the Crow Creek watershed and areas in the immediate vicinity of municipal wells and wells, if any, developed for project construction in the DA.

2.4 Derivation of Data Base

This section describes the data collection related to water resource assessment. Primary sources of data for the description of the existing water resource system include: United States Environmental Protection Agency (EPA); U.S. Geological Survey (USGS); U.S. Forest Service (USFS); Soil Conservation Service (SCS); Wyoming State Engineer's Office (WSEO); Wyoming Department of Environmental Quality (WDEQ); Wyoming Water Research Center; Wyoming Water Development Commission; Nebraska Conservation and Survey Division; Nebraska Department of Health (NDH); Nebraska Department of Water Resources (NDWR); Nebraska Department of Environmental Control (NDEC); F.E. Warren [illegible]; Cheyenne Board of Public Utilities (CBPU); the City of Cheyenne, and other municipal reports, data files, and interviews.

A separate water acquisition program for the project is being developed in conjunction with the NDWR, CBPU, and the WSEO that will describe methods and procedures to acquire water for project construction and operation (Ertec Western, Inc. 1983a, 1983b). Data collected as part of that effort were utilized for the water resources environmental analysis.

2.4.1 Literature Sources

The water resource system in the ROI nas been extensively studied and several reports have been written summarizing these studies.

Key data sources used to describe existing groundwater and surface water hydrology and quality include:

- o WATSTOR and STORET for water quality data. These data storage programs are available from USGS and EPA and provide histories of water quality and quantity data in the ROI;
- o USGS reports on the groundwater situation in the various counties and watersheds;
- o Reports gathered from local agencies on water use and facilities;
- o The Wyoming Water Research Center Water Resource Data System and studies on various groundwater basins;
- o Environmental and design studies done for the Cheyenne Stage II projects;
- o USGS reports on Crow Creek and groundwater studies by USGS in Laramie County;
- o WSEO, NDWR, and Ertec Western, Inc. for water rights data based on computer printouts and legislative documents; and
- o Flood control studies for Cheyenne and Laramie County done by the Corps of Engineers (COE) and Federal Emergency Management Agency (FEMA).

A complete list of all references reviewed is included in Section 5.0, References Cited and Reviewed.

2.4.2 Group and Agency Contacts

Data were needed on the existing raw water supplies for each municipality in a deployment county, as well as existing water use in those municipalities. This required field visits, phone calls, and literature acquisition in Platte, Goshen, and Laramie counties, Wyoming, and Banner, Kimball, and Scotts Bluff counties, Nebraska.

Several meetings, contacts, and discussions took place with Nebraska, Wyoming, and Cheyenne officials as water demand values were calculated. Sufficient data were gathered from literature review to describe existing conditions and future trends in the ACSs and ROI for nonmunicipal water users.

Several meetings were held in the ROI to collect data or review proposed analytic methodologies. These included meetings with staff of the WSEO, WDEQ, Wyoming Water Development Commission, Nebraska Conservation and Survey Division, NDWR, NDH, NDEC, F.E. Warren AFB, and the CBPU during the prescoping process and initial data analysis. Meetings with USGS personnel were also held to interpret past work on water resources done in the ROI. The public scoping process provided additional opportunities for comment on data base availability and applicability of proposed analysis methodologies.

Several coordination meetings were also held with Ertec Western, Inc. and COE officials to assess water demands and plans for water supply.

2.4.3 Primary Data

No field collection of primary data was conducted for this report outside of the Crow Creek watershed. A new assessment of the water supply capability of the Crow Creek watershed was required and evaluated in conjunction with Ertec Western, Inc. (Earth Technology Corporation). Field study efforts were required to define Cheyenne wellfield production capability, sources and fate of raw water losses in the existing Cheyenne raw water delivery system, use of water, and contractual arrangements for nonmunicipal use of the raw water supply. Statistics on rainfall runoff-storage features of the watershed were required. Existing groundwater and surface water hydrology and their interrelationships in this watershed were needed to describe water availability, present water supply facilities, and the present state of development for the available supply.

Field data on aqueducts, wellfields, reservoirs, and existing water use and diversions were gathered to allow an assessment of the existing water resource patterns in the watershed.

Stormwater effects are only expected to be significant in the Cheyenne Urban Area. Field programs necessary to set up and run various models were needed to evaluate this effect. This consisted of delineation of drainage basins and characterization of existing drainage routing and structures.

Field reconnaissance of the Crow Creek watershed and the existing drainage system in the Cheyenne Urban Area was required. This included limited field data on channel cross sections and characteristics, existing drainage structure dimensions and control facilities, and floodplain elevation supplements to existing secondary data for stormwater analysis.

2.5 Analytic Methods for Existing Conditions

This section describes analytic methods used to describe existing conditions. Summaries of some methods are contained in Appendix A.

2.5.1 Water Use and Demand

The methodology used to describe existing water use patterns in the Crow Creek watershed was to review existing reports and develop unit water demand factors for different types of users. Municipal users include the Cheyenne service area and other municipal systems. Nonmunicipal users include residential, commercial, industrial, agricultural irrigation, and livestock water consumers not supplied by municipal systems.

Existing water supply sources and their capability to meet water demands were summarized based on existing data and water development plans and programs. Where the capability of the water supply source could not be determined, primary data were developed to make such a determination.

Existing water use in the other areas of the ROI was summarized based on secondary data obtained.

2.5.2 Constraints on Water Use

The process for obtaining water rights, and existing legal restrictions relative to water use and water quality degradation were summarized. Existing groundwater and surface water rights were also summarized to provide a basis to determine general probability of impact.

Existing water quality criteria and standards were also summarized for Wyoming and Nebraska because they provide legal requirements to be met by project facilities.

2.5.3 Surface Water Hydrology and Quality

Existing surface water flow and quality records were reviewed and summarized for the Crow Creek watershed. Average values for flow and quality were calculated. Existing flood records were summarized. From primary and secondary data, a delineation of drainage areas and characteristics of flow channels for each contributing watershed area was made.

The impacts of existing wastewater discharges on Crow Creek were summarized based on results of water quality assessments performed to set effluent limits for Cheyenne (WDEQ 1979).

Existing flood potential in the Crow Creek watershed has been determined using HEC-1 and HEC-2 computer programs developed by the COE (U.S. Army Corps of Engineers 1966, 1973) as well as a computer program developed by the Soil Conservation Service (SCS) called TR-20 (USDA 1975).

A new assessment of flood potential was made as part of this study using SCS methods (SCS 1975a, 1975b) and EPA methods (EPA 1982) for the urban area. An additional computer flood routing method was used to route flows through the Crow Creek Basin.

For the other ACSs, the data from WATSTOR and STORET were summarized and a map was prepared to show locations of flow and quality monitoring stations. Ranges and average values of flow and quality for each station were tabulated. Drainage areas of the various watersheds were calculated and a gross assessment of existing erosion and sedimentation potential from current land uses was made assuming typical erosion rates for cropland and grassland.

2.5.4 Groundwater Hydrology and Quality

Existing groundwater flow patterns, regional aquifer characteristics, and water quality data were summarized for the ACSs. These data are not sufficient to calculate site-specific changes in groundwater levels from project wells and will have to be supplemented by primary data collection near a proposed project well. Data collection could include a program to:

- Conduct site-specific pump tests to define aquifer characteristics;
- Determine the magnitude of water level declines and water chemistry changes at project wells, existing wells, and local springs; and
- Determine the effects of water withdrawal on stream discharge.

Groundwater budgets were developed for the Crow Creek watershed based on existing data. The regional impact of withdrawals could be assessed relative to quantitative changes in the budget.

A simplified mass balance equation was used in conjunction with the groundwater budget to provide a measure for potential impacts on groundwater quality of wastewater discharges from the Cheyenne Urban Area.

It was determined that existing data were insufficient to describe the site-specific impacts of operation of the Cheyenne wellfields and a program to set up and run a modification of the USGS two-dimensional groundwater model was initiated by the Earth Technology Corporation related to water supply development planning. Results of that effort will be used to better quantify impacts prior to project construction.

2.6 Existing Environmental Conditions

2.6.1 Water Use and Demand

2.6.1.1 Crow Creek Watershed

2.6.1.1.1 Municipal Water Use

2.6.1.1.1.1 Cheyenne

Cheyenne has three sources of water supply. Reservoirs in the Crow Creek watershed west of Cheyenne provided an average net yield of approximately 4,600 acre-ft/yr from the Crow Creek watershed in the 1976 to 1982 period and records indicate a long-term average net yield of about 5,000 acre-ft/yr (USFS 1981). The reservoirs and their approximate storage capacities are: North Crow Reservoir, 1,865 acre-ft; Granite Springs Reservoir, 5,321 acre-ft; and Crystal Lake Reservoir, 3,410 acre-ft (Banner Associates 1983a). According to the CBPU, the usable capacity of Crystal Lake Reservoir is currently limited to less than its normal capacity pending the outcome of an investigation of the dam's stability by the COE. A current agreement with the Wyoming State Engineer allows 2,962 acre-ft of storage in Crystal Lake (Banner 1983a). During periods of low precipitation, such as 1975 to 1977, the yield of the Crow Creek water supply is not reliable and the CBPU must rely more heavily upon its other supplies or reduce the amount of water stored in the Crow Creek reservoirs.

One additional source of water for Cheyenne is groundwater. The municipal wellfields (Federal, Bell, and Happy Jack) consist of 44 wells, located west and northwest of the city. These wells, in the Ogallala and White River aquifers, have provided the city with about 3,400 acre-ft/yr on a long-term average basis and 3,300 acre-ft/yr in 1976 to 1982. Because of recent declines in groundwater levels in the area, the CBPU has elected to use these wells only during peak demand periods and has attempted to limit withdrawal rates for municipal use to about 2,000 acre-ft per year to maintain water table elevations at current levels (USFS 1981). However, during a recent drought period the wellfields provided approximately 8,000 acre-ft of water in 1977.

The well system is generally used to meet peak summer demands because of treatment capacity limitations. During the period of peak summer use, the

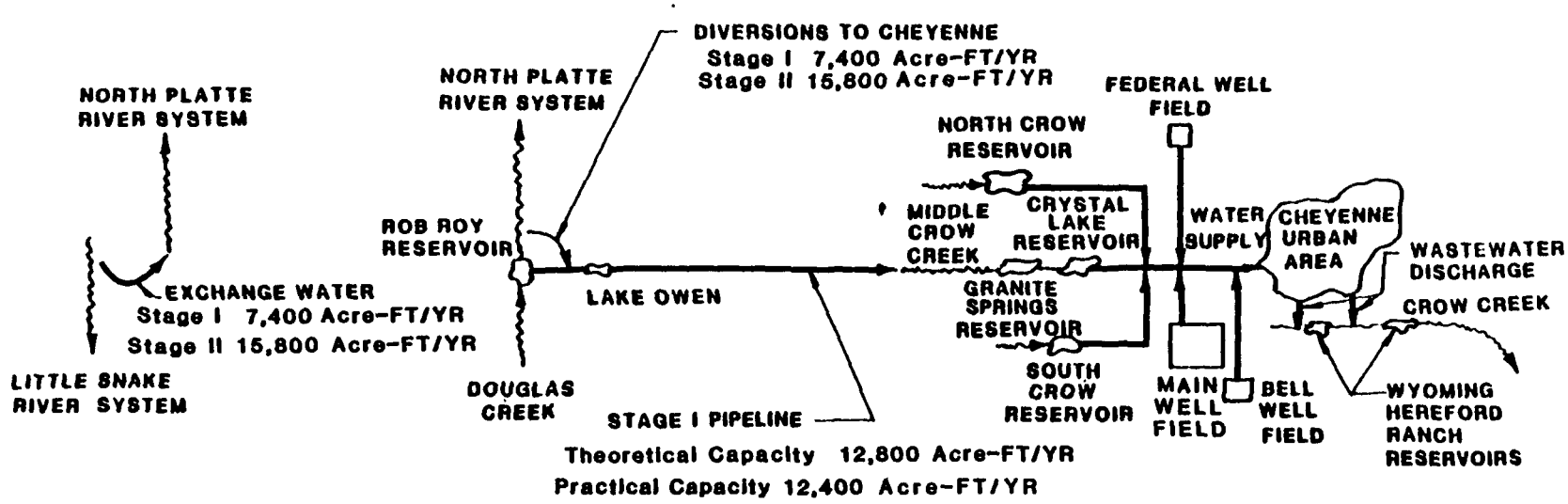
wellfields are used to produce as much as 9 million gallons per day (mgd) (Banner 1983a). Water from the wellfields requires only chlorination before its use.

Contractual obligations require the CBPU to provide a portion of the water from the municipal wellfields for agricultural irrigation. For the period 1976 to 1982 this has averaged approximately 500 acre-ft/yr.

Because Cheyenne needed a reliable surface water supply for its growing population, the CBPU developed the Douglas Creek watershed in the North Platte River Basin (referred to as the Cheyenne Stage I Water Diversion Project). This diversion of Douglas Creek water via a pipeline to the Middle Branch of Crow Creek (and Granite Springs and Crystal Lake reservoirs) requires the replacement of a like amount of water to the North Platte Basin from water developed by the CBPU in the Little Snake River Basin west of the Continental Divide. A schematic of the physical system to deliver this water is shown on Figure 2.6-1. The current capacity of this system is limited by the amount of replacement water available from the Little Snake River Basin. The average water transferred from the western slope in the period of 1976 through 1982 has been 7,230 acre-ft/yr. Because it is not possible to exactly balance the amount of water conveyed into the North Platte system with the amount diverted from it, actual transfers of water to the Crow Creek watershed are somewhat less than this amount. In 1976 through 1982, approximately 6,600 acre-ft/yr were diverted from Douglas Creek to Crow Creek, of which about 6,140 acre-ft/yr reached Granite Springs Reservoir. The difference is due to evaporation, seepage, and transmission losses upstream of Granite Springs Reservoir. The excess diversions indicated by the above are common operating practices in order to assure that no deficit is created in the North Platte system. A portion of the excess diversions is used to provide water to Pacific Power and Light (PP&L); these have averaged 240 acre-ft/yr for 1976 to 1982.

Construction is currently under way to enlarge the collection capability of the Little Snake and Douglas Creek watershed facilities. Because of this construction the current capacity is reduced; however, following completion of construction in 1985, both watersheds will have developed capacities of approximately 23,000 acre-ft/yr. Diversions from Douglas Creek will then be limited by the capacity of the pipeline from Lake Owen to Crow Creek. The theoretical capacity of this pipeline is 12,800 acre-ft/yr, but the practical delivery capacity is currently between 11,000 and 11,400 acre-ft/yr (Wyoming Department of Economic Planning and Development January 1983). Following completion of construction it is estimated that 12,400 acre-ft/yr can be diverted from Douglas Creek to the Crow Creek watershed (CBPU May 1983). At that time some of the constraints on operation of the delivery system will have been removed. Average water use and supply values from these various sources are summarized in Table 2.6-1.

The CBPU supplies water to the city of Cheyenne, F.E. Warren AFB, and the South Cheyenne Water & Sewer District (SCW&SD). The water use in 1982 for the CBPU service area was 13,500 acre-ft; this compares to an average annual water



NOTE:

Diversions to Cheyenne from North Platte River System cannot exceed exchange water from the Little Snake River System to the North Platte River System.

SCHEMATIC OF WATER RESOURCE SYSTEM FOR CHEYENNE URBAN AREA

FIGURE NO. 2.6-1

Table 2.6-1

CHEYENNE MUNICIPAL WATER USE AND SUPPLY
1976-1982

	acre-ft/yr
Water Use	13,020
Water Supply	
Groundwater	3,320
Surface Water	9,700
North Crow	1,220
Middle Crow[1]	8,480

Note: 1 Middle Crow includes water collected from the Crow Creek drainage as well as import water from Cheyenne Stage I. Import water and total outflow from the reservoirs to Cheyenne are metered. The CBPU calculates the water collected from the Crow Creek drainage by the difference in these values. This assigns all reservoir evaporation and spillage losses to Crow Creek drainage when in fact part of the evaporation and spillage losses are from import water. Considering this fact, the real net water supply from the Middle Crow Creek drainage is about 3,380 acre-ft/yr and from Stage I a net water supply of 5,100 acre-ft/yr from a net diversion of 6,600 acre-ft/yr.

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use in the period 1976 to 1982 of 13,020 acre-ft. The distribution of the water use in the Cheyenne service area is shown in Table 2.6-2. The year 1980 is shown here because it is the most recent year for which complete records by type of user were readily available. Although 1980 water use was somewhat higher than other recent years, the distribution of use by type of user seems to be representative. The one significant variation in Table 2.6-2 from actual 1980 use data occurs in the "Civic" category which is defined as including irrigation of parks, golf courses, and cemeteries, street cleaning, firefighting, distribution losses, etc. Actual 1980 "civic" use for Cheyenne was approximately 2,300 acre-ft while the average for 1977 to 1980 was 2,000 acre-ft/yr. The latter figure was also used by the WSEO in analyzing the Stage II Water Diversion Project and is used here as a representative value. Of the "civic" use component, an estimated 850 acre-ft/yr is used for irrigation of golf courses, parks, and cemeteries (Cheyenne Water Conservation Advisory Group 1982). Approximately 400 acre-ft/yr of this water is used to irrigate the Airport Golf Course.

Cheyenne has a fairly stable industrial base which uses about 2,200 acre-ft of water annually. Currently this constitutes about 20 percent of the total CBPU-supplied water. As residential population and its associated water uses increase, it is expected that the industrial use will remain fairly constant (USFS 1981). For the city of Cheyenne, nonindustrial use amounts to about 194 gpcd; for the entire Cheyenne area the per capita nonindustrial use is approximately 180 gpcd due to somewhat lower nonindustrial use rates in SCW&SD (88 gpcd) and F.E. Warren AFB (160 gpcd).

Table 2.6-2

CHEYENNE MUNICIPAL WATER USE[1] IN 1980

Area	Population	acre-ft/yr Residential	Commercial	Civic[2]	Industrial	Total
Cheyenne	47,283	6,320	1,990	2,000	2,200	12,510
South Cheyenne[3]	6,200	580	10	20	-	610
F.E. Warren AFB	3,627	375	50	225	420	1,070
TOTAL:	57,110	7,275	2,050	2,245	2,620	14,190

Notes: 1 Reflects finished water supplied.

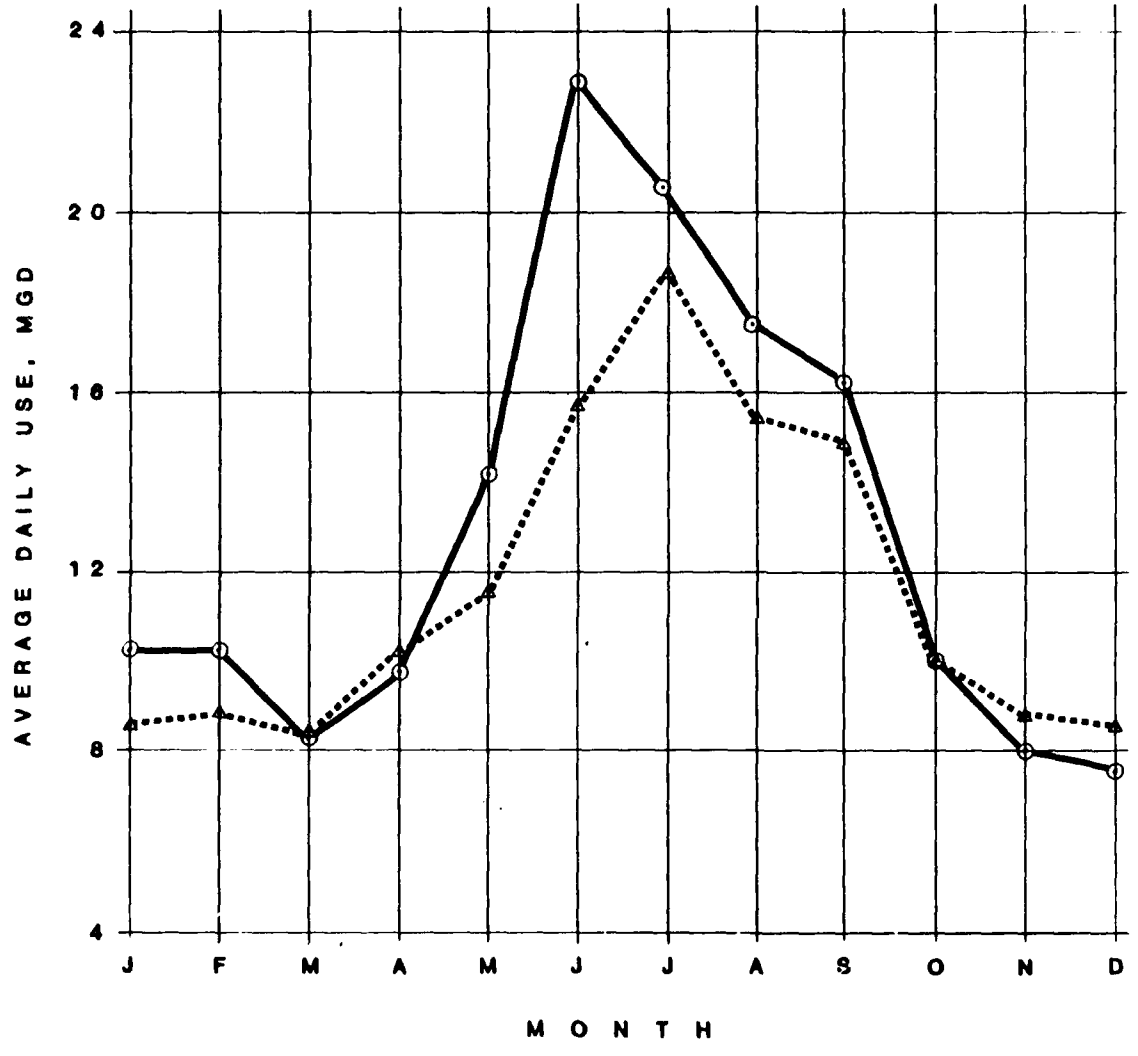
2 Includes irrigation (parks, golf courses, etc.), firefighting, distribution losses, etc.

3 Includes only water supplied by the CBPU.

The seasonal pattern of water use in Cheyenne is generally typical of that in most semiarid regions. The peak usage period occurs in the summer, generally in July when large quantities of water are required for lawn watering. Water use on the day of maximum demand is approximately 2.3 times the average daily use based upon the 1976 to 1982 average. In the last few years peak daily use has been somewhat higher, 2.5 times average use, based upon the 1980 to 1982 average. The highest daily water use recorded in Cheyenne was 31.5 mgd in 1980. Residential, commercial, and civic uses all tend to have their peak demands concurrently in the summer while the maximum industrial use generally occurs in the fall and early winter. These variations in demand are shown on Figure 2.6-2.

2.6.1.1.1.2 South Cheyenne Water & Sewer District

The SCW&SD is a special utility district which serves an unincorporated area south of, and immediately adjacent to, the city of Cheyenne. This area is primarily residential in nature with the majority of the population residing in mobile home courts or on single lots. Prior to 1971, residents relied on wells in the area for their water supply. In 1971, primarily because of contamination of local wells, the District connected its water distribution system to Cheyenne's and began purchasing water from the CBPU; this water is resold to customers in the SCW&SD. In 1980, the SCW&SD purchased approximately 610 acre-ft of water from CBPU. In 1982 this rose to 670 acre-ft, reflecting continued growth in the SCW&SD service area. In addition to the water purchased from the District, many residents continue to use private wells for irrigation and/or domestic use. The actual amount of water used from private



SEASONAL WATER USE PATTERN IN CHEYENNE

FIGURE NO. 2.6-2

wells is difficult to estimate accurately. The lower per capita residential use in the SCW&SD as compared to the city of Cheyenne (84 gpcd vs. 119 gpcd) is probably due largely to the continued use of private wells. This use of private wells is estimated as the difference between the per capita residential rates for the 2 areas, which is approximately 35 gpcd or 250 acre-ft/yr.

The seasonal use patterns for CBPU-supplied water in the SCW&SD parallel those in Cheyenne. Peak demand occurs in July or August.

2.6.1.1.2 Industrial and Commercial Water Use

2.6.1.1.2.1 Industrial Water Use

F.E. Warren AFB is billed as an industrial user by the CBPU; however, because this use is not primarily industrial in nature it is discussed in a separate section.

Industrial water users can be placed in two categories within the watershed. The first group contains those industries which are supplied by the CBPU. The major water-using industries in this group are the Husky Oil Refinery, which uses approximately 1,800 acre-ft annually; the Union Pacific Railroad, which uses approximately 270 acre-ft/yr; and Wyott Manufacturing, which uses 110 acre-ft/yr. Husky Oil and the Union Pacific have their own wastewater treatment systems.

Industrial use is more constant than the other types of use in the Cheyenne area. The peak monthly industrial use is about 25 percent higher than the average monthly industrial use. The peak use generally occurs in fall or early winter. Industrial water demand from the CBPU system has been relatively constant in recent years and is anticipated to change only slightly in the near future (USFS 1981).

The second group includes industries that supply their own water. The only self-supplied industrial water user in the Crow Creek ACS is the Wycon Chemical Company, located approximately 10 miles west of Cheyenne. Wycon used approximately 870 acre-ft annually from its 19 wells in the Ogallala aquifer in 1980 (Libra et al. 1981) with more recent use of 400 acre-ft/yr from 4 wells (Wycon 1983).

2.6.1.1.2.2 Commercial Water Use

Commercial water users, like the industrial, can be grouped into those supplied by the CBPU and those with their own supplies. The CBPU supplies approximately 2,000 acre-ft/yr for commercial users. The largest of these is the Little America Motel and Truck Stop complex on the southwest edge of Cheyenne which uses approximately 120 acre-ft/yr from the CBPU system. In addition, Little America has its own well producing approximately 47 acre-ft/yr from the Ogallala aquifer (Libra et al. 1981). Cheyenne's commercial water demand has a seasonal pattern similar to residential water use in the City.

Other commercial water users with their own water supplies include numerous motels, cafes, and campgrounds in the Cheyenne urban fringe area and throughout the watershed. Most of these establishments use small quantities of

groundwater from relatively shallow wells and constitute a total annual demand of about 53 acre-ft (Libra et al. 1981). Use for these establishments is probably concentrated in the summer months.

2.6.1.1.3 Agricultural Water Use

2.6.1.1.3.1 Irrigation

Irrigation of cropland constitutes the single largest water demand in Laramie County. Both ground and surface water are used, with the majority of the irrigation occurring in the eastern portion of the county, particularly in the Pine Bluffs area. There were approximately 46,400 acres of irrigated crops in Laramie County in 1981 (Wyoming Crop and Livestock Reporting Service 1982), the majority of which were hay (62%).

Based upon aerial photographs and land use data it is estimated that 11,300 acres within the Crow Creek watershed ACS are irrigated. Of this, 6,600 are irrigated with groundwater and 4,700 with surface water. Total water used for irrigation is estimated at 9,400 and 7,000 acre-ft/yr for ground and surface water, respectively. Irrigation withdrawals from groundwater are seasonal in nature with the highest applications during the summer months. Surface flows are diverted during peak runoff periods and are used for irrigation until water supply runs short (late July or August). Much of the land irrigated with surface water also receives supplemental irrigation from groundwater. The amount of supplemental water applied varies depending upon the amount of surface water available in a particular year.

2.6.1.1.3.2 Livestock

Another agricultural use of water is for livestock watering. There are approximately 60,000 cattle and 29,000 sheep in Laramie County (Wyoming Crop and Livestock Reporting Service 1982). Based on land use patterns, it is estimated that approximately 25 percent of these are located in the Crow Creek watershed. Based upon average water consumptions of 15 and 3 gallons per animal per day, respectively, the annual use for livestock watering would be about 280 acre-ft/yr within the watershed. It is estimated that about 80 percent of this (220 acre-ft) is supplied from groundwater. Livestock water use occurs year-round; however, peak demands occur during the summer months.

2.6.1.1.4 F.E. Warren AFB Water Use

F.E. Warren AFB receives its water supply from the CBPU. Annual consumption is approximately 1,070 acre-ft. Residential use accounts for about 375 acre-ft of this, for both onbase family housing and barracks. Another 225 acre-ft annually is used for irrigation of the base's golf course and parade grounds (Baker, Sweeney Associates 1981). The remaining amount (470 acre-ft/yr) is used for commercial and industrial purposes. Because of the high proportion of water used for irrigation, seasonal patterns of use on F.E. Warren AFB are similar to those in Cheyenne; however, the ratio of peak-to-average demand is significantly higher. Maximum monthly use of over 2 times the average monthly use occurs in July, as opposed to a maximum of 1.6 times the average use for Cheyenne. Minimum monthly use occurs during the winter period with demand being about 40 percent of the average daily use. Water use, like the base

population, has been relatively stable in recent years as shown by water use records of the CBPU.

2.6.1.1.5 Other Water Uses

The town of Carpenter is also in the Crow Creek watershed but does not have a central water system. In addition to the previously discussed uses, water is also used by individuals living in mobile home parks with central water systems, rural subdivisions and communities without central water systems, and on farms and ranches. There were about 6,800 such people in the Crow Creek watershed in 1980, 1,480 of which lived in mobile home courts. Water use for these individuals is estimated at about 680 acre-ft/yr. Table 2.6-3 provides a breakdown of existing surface and groundwater use in the Crow Creek watershed.

2.6.1.1.6 Existing Water Development Plans

Surface water in the Crow Creek watershed is considered to be fully appropriated. Therefore, any future surface water supplies to be developed must involve the diversion of water from other drainage basins, as in Cheyenne's Stage I Project. Groundwater is heavily utilized in eastern Laramie County and water table levels have experienced seasonal declines. As a result, a groundwater control area has been established by the Wyoming Board of Control in eastern Laramie County which includes the eastern portion of the Crow Creek watershed. Any future water development projects must consider these two factors.

Water development planning for the Crow Creek ACS has included consideration of upgrading reservoirs and constructing new facilities (Banner 1983b), but results have not led to recommendations for implementation due to high cost-benefit ratios and uncertainty of the capability of the existing supply.

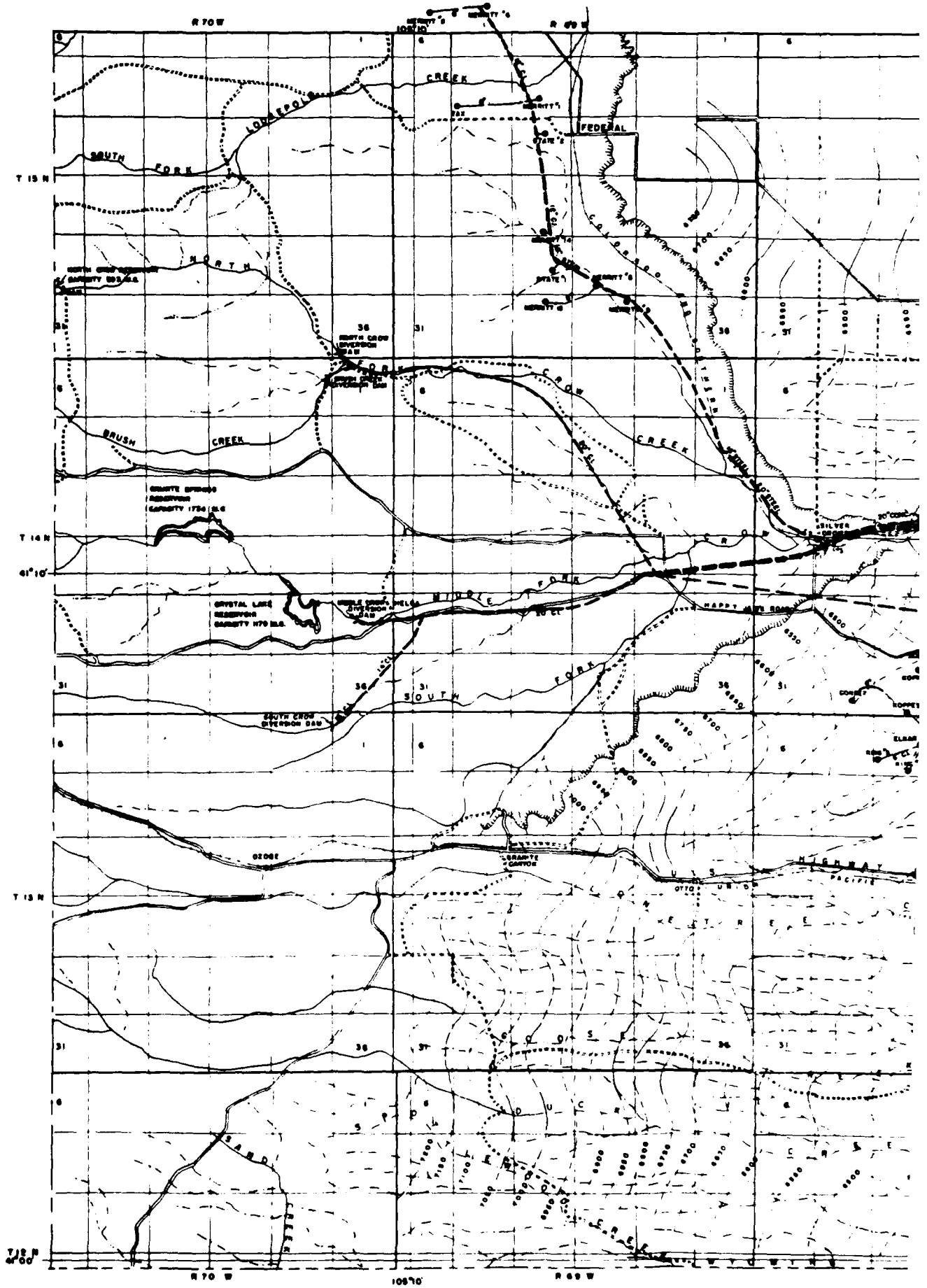
2.6.1.1.6.1 Capability of Existing Supply

Crow Creek water supply is obtained from the North, Middle, and South forks of Crow Creek and from Brush Creek, a tributary of the North Fork of Crow Creek. For some basins in the watershed, specifically the South Fork of Crow Creek and Brush Creek, little or no storage exists and streamflow must be diverted when available in order to be used. On other streams, storage facilities allow for more efficient use of available water. Storage facilities include the North Crow Reservoir and the North Crow Diversion Reservoir on the North Fork; Granite Springs and Crystal Lake Reservoirs on the Middle Fork; and the South Crow Diversion Reservoir on the South Fork of Crow Creek. Total raw water storage capacity in the Crow Creek watershed is about 10,260 acre-ft (Banner 1983a).

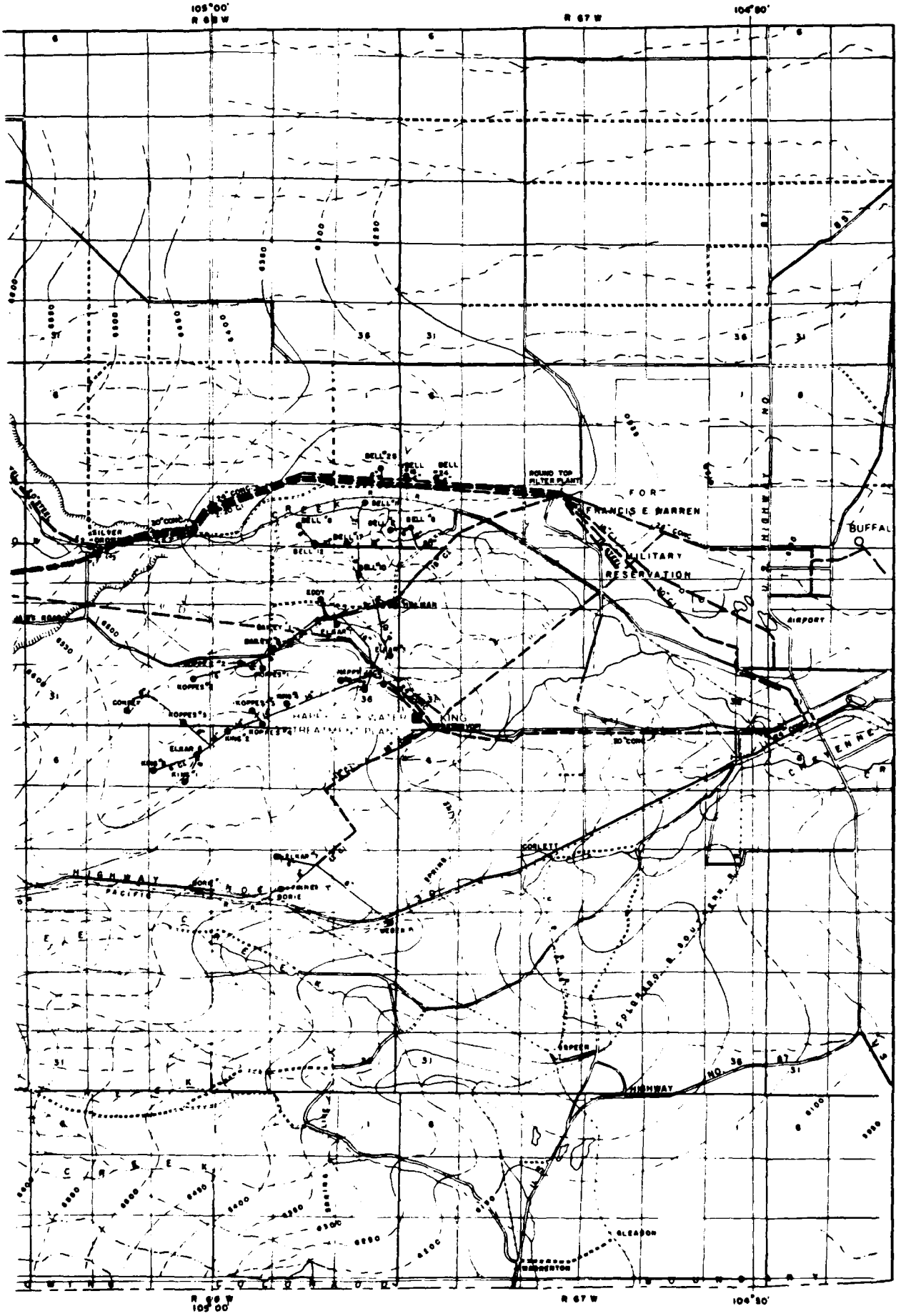
Figure 2.6-3 shows Cheyenne water supply facilities in the Crow Creek drainage area.

The potential yield of the Crow Creek watershed is related to total precipitation falling in the watershed. A long-term record of precipitation is available for Cheyenne Municipal Airport, with over 40 years of record. Other precipitation gages in the watershed provide data for limited periods of time, typically 5 to 15 years. Using 3 gages at elevations of 6,126, 7,930, and

(1)



CHEYENNE WATER SUPPLY FACILITIES CROW CREEK



(2)

LEGEND

- WELL
- SPRING
- CONTOUR LINE ON GROUND WATER SURFACE
- SPRING LINE
- PIPE LINES
- COLLECTION PIPE LINES

NOTE

BASE MAP FROM A MAP DATED [illegible] BANNER ASSOCIATES, INC.

NOTE

Groundwater surface contours obtained from USGS "Progress Report on Geology and Ground Water Resources of the Cheyenne Area, Wyoming" (1962)

SCALE ½" = 1 MILE

OW CREEK DRAINAGE AREA

FIGURE NO. 2.6-3

Table 2.6-3

EXISTING WATER USE - CROW CREEK WATERSHED
1980
(acre-ft/yr)

	Ground-water	Surface water	Total
Municipal[1]	3,320	9,700	13,020[a]
Industrial/Commercial[2]	970	-	970
Irrigation	9,400	7,000	16,400
Livestock	220	60	280
Domestic (nonmunicipal)[3]	930	-	930
TOTAL:	14,840	16,760	31,600

Notes: 1 Includes Cheyenne, South Cheyenne, and F.E. Warren AFB.

2 Includes only self-supplied users.

3 Includes private wells in South Cheyenne.

a Based upon 1976-1982 average use values for Cheyenne.

8,170 feet for the period of 1976 to 1982, an approximate relationship between elevation and precipitation was found. These records indicate an increase in precipitation of about 2.8 inches for each 1,000 foot increase in elevation.

This relationship was used to determine precipitation in the watershed based upon mean annual precipitation in Cheyenne for 1976 to 1982. This period was selected due to compatibility with water supply records and close agreement between the 1976 to 1982 annual precipitation of 14.6 inches and the longer-term 1941 to 1982 annual precipitation of 14.43 inches for the Cheyenne Municipal Airport gage. Table 2.6-4 shows the precipitation volumes determined for the basins in the Crow Creek watershed.

Streamflow data for a gage on the Middle Fork above Granite Springs (1941-1963) and South Fork above the diversion reservoir (1951-1969) provide a method to determine runoff rates for those basins. Precipitation volumes were calculated for the corresponding years based upon Cheyenne precipitation and the elevation relationship. Average runoff rates were determined by comparison of these volumes with gaged streamflow volumes. For the Middle Fork above Granite Reservoir, a runoff rate of 10.8 percent was determined, and a runoff rate of 6.9 percent was found for the South Fork. Due to comparable physical characteristics, the North Fork above the North Crow Reservoir was assumed to yield the same runoff rate as the Middle Crow above Granite, or 10.8 percent. All other drainages were considered comparable to the South Fork, and 6.9 percent was used.

Table 2.6-4

CROW CREEK WATERSHED WATER SUPPLY CAPABILITY

Basin	Surface Area (acres)	Average Elevation (ft)	Total Precipitation Volume (acre-ft)	Runoff Rate (%)	Total Available Flow (acre-ft)
North Crow Creek	29,100	7,670	45,800	-	3,840
Above North Crow Reservoir	10,300	8,150	17,300	10.8	1,870
Brush Creek	10,300	7,470	15,700	6.9	1,090
North Crow Creek between Reservoir and Diversion	8,500	7,320	12,800	6.9	880
Middle Crow Creek	26,300	7,850	42,500	-	4,070
Above Granite Reservoir	17,800	8,000	29,300	10.8	3,160
Below Granite Reservoir but above Diversion	8,600	7,520	13,200	6.9	910
South Crow Creek	9,600	7,650	15,100	6.9	1,050
TOTAL Crow Creek above Diversions:	64,900	7,750	103,400	-	8,960
TOTAL Crow Creek below Diversions:	124,600	6,450	160,900	1.0	1,610

Total available flow for the Crow Creek watershed above diversions is about 8,960 acre-ft/yr, based upon the runoff rates, 1976 to 1982 precipitation in Cheyenne, and the precipitation elevation relationship. The results are summarized in Table 2.6-4. The North, Middle, and South Fork basins contribute 3,840, 4,070, and 1,050 acre-ft, respectively.

Water system operation records for 1976 to 1982 (CBPU 1983) provide measurements of contributions from groundwater and Stage I diversion. Water diverted for water supply and irrigation use is also provided.

Using this information and streamflows calculated above, a representation of average system operation is provided in Figure 2.6-4. Evaporation losses are determined using area-volume relationships for the reservoirs, storage volume records, and evaporation rates developed for these facilities (Banner 1974). Evaporation losses in Crow Creek total 880 acre-ft. Volumes attributed to spillage and losses are calculated to balance inflow and outflows. The total volume of water in this category is 4,520 acre-ft and is assumed to bypass diversion facilities.

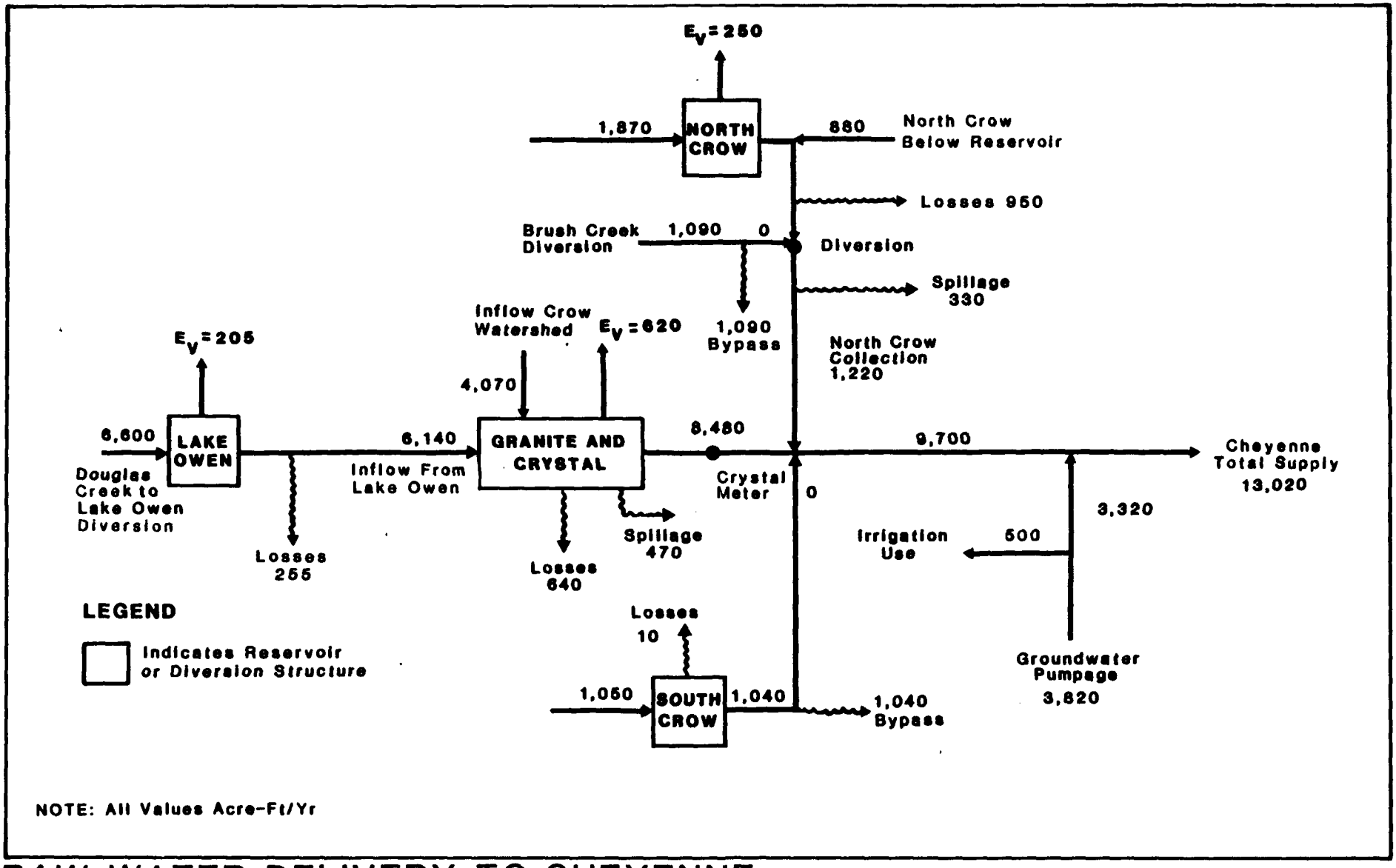
For the period 1976 to 1982, 8,960 acre-ft of water reached storage facilities or diversion points in the Crow Creek watershed. Of this, over 3,400 acre-ft bypassed diversion points on the North and South forks without being collected. Evaporation losses from reservoirs on those streams constituted an additional 260 acre-ft. On the Middle Fork, evaporation and spillage losses totaled 1,730 acre-ft. Because Stage I diversions enter this system, the watershed contributes only 40 percent of the flow in this system. Using this fraction of losses, 690 acre-ft of losses in the Middle Fork are attributed to Crow Creek. The total supply from the Crow Creek watershed is seen to be 8,960-3,410-260-690 = 4,600 acre-ft. This figure differs from those of the water supply records (CBPU 1983) in that all Middle Crow losses are attributed to the Crow Creek watershed in those records. For this study, the amount of water recorded at the Crystal meter (8,480 acre-ft) due to inflow from the Middle Crow watershed is 3,380 acre-ft, which is the inflow (4,070 acre-ft) less Middle Crow share of losses:

$$\left[\frac{4,070}{6,140 + 4,070}\ (640 + 470 + 620)\right] \quad \text{or 690 acre-ft}$$

The amount from Stage I at the Crystal meter is 5,100 acre-ft (8,480 - 3,380).

Nearly two-thirds of the area of the Crow Creek watershed above the Cheyenne streamflow gage is located below the diversion points. This 124,600 acres is shown in Table 2.6-4 to receive a total of 160,900 acre-ft in precipitation. A runoff rate must be assumed for this basin, and 1 percent is used to estimate streamflow. This yields 1,610 acre-ft in available flow. In addition, 4,520 acre-ft in spillage and bypass enter the stream at the diversion points. The total flow below the diversions is therefore 6,130 acre-ft.

The documented historical mean annual flow at the Cheyenne stream gage is 12.1 cubic feet per second (cfs). Of this flow, 6.5 cfs is attributed to sewage treatment plant effluent, leaving 5.6 cfs, or 4,050 acre-ft, in streamflow. This results in a net loss of 2,080 acre-ft in streamflow between the diversions and the Cheyenne gage, presumably to groundwater infiltration.



RAW WATER DELIVERY TO CHEYENNE

FIGURE NO. 2.6-4

The values indicate that a net of about 4,600 acre-ft is collected from the watershed's potential supply of about 7,800 acre-ft (8,960 inflow less an estimated 1,160 acre-ft evaporation loss if all were collected). Therefore, an unused potential of about 3,000 acre-ft/yr exists in Crow Creek watershed. Water from Brush Creek and the South Fork of Crow Creek have some water quality problems at certain times of the year indicating water treatment problems in using this supply.

Capturing more of the surface runoff in reservoirs for municipal use could result in reduced recharge to the groundwater basin from which municipal wells obtain their supply. The capacity for increased yield from existing surface water supplies may therefore be offset to some extent by reductions in groundwater recharge.

The above analysis describes the Crow Creek watershed's capability in an average year. However, planning of water system operations and analysis of project impacts must consider the possibility of reduced precipitation during the project period. For this reason precipitation records were analyzed to determine the reduction in runoff to be expected for a drought with a 10 percent probability of occurrence. Statistical analysis of National Oceanic and Atmospheric Administration (NOAA) precipitation records for Cheyenne from 1942 through 1981 indicates that a 1-in-10 year drought corresponds to an annual precipitation of 9.77 inches, as compared to an average annual precipitation of 14.4 inches.

Table 2.6-5 shows the effect of this reduced precipitation on the expected runoff in the Crow Creek watershed using the previously discussed methodology. The total reduction in runoff above diversions for this drought is 2,260 acre-ft; however, since the Brush Creek and South Crow diversions are currently not utilized, the effective reduction in runoff above the diversions currently used for water supply is 1,700 acre-ft.

The reduction in water supplied under drought conditions is reflected in Figure 2.6-5. This figure shows average 1976 to 1982 conditions with the reduced runoff rates shown in Table 2.6-5. The net effect is a decrease in water supply of 900 acre-ft. This is based upon the following assumptions regarding spills and losses. Losses are assumed to remain at the 1976 to 1982 average values during drought conditions. These are generally losses to the streambed while water is flowing between reservoirs (North Crow to North Crow Diversion and Granite Springs to Crystal Lake). Spills, which are losses over reservoir spillways which are not recaptured downstream, are assumed to be zero during drought conditions due to the decreased amount of runoff. This appears to be consistent with CBPU operation records for these reservoirs.

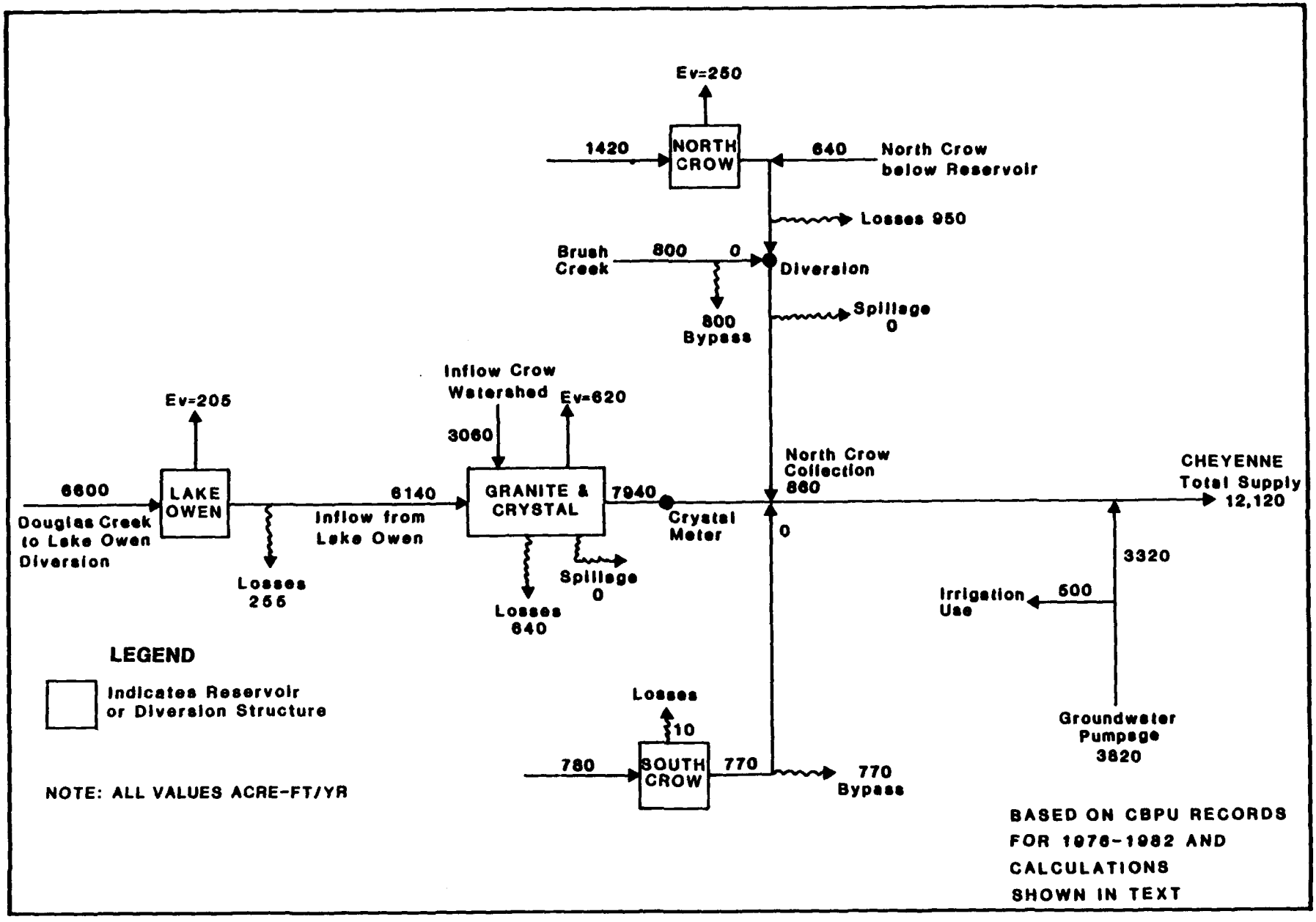
This analysis indicates that for a drought with a 10-percent probability of occurrence, which corresponds to a 32-percent (4.6 inches) decrease from the average annual precipitation, the Crow Creek watershed will yield approximately 20 percent less (900 acre-ft) than in an average year. This 900 acre-ft reduction in water supply will be used in analyzing the CBPU water supply for both baseline conditions and project impacts.

Table 2.6-5

CROW CREEK WATERSHED CAPABILITY, DROUGHT CONDITIONS[1]

Basin	Surface Area (acres)	Average Elevation (ft)	Total Precipitation Volume (acre-ft)	Runoff Rate (%)	Total Available Flow (acre-ft)
North Crow Creek	29,100	7,670	34,071	-	2,860
Above North Crow Reservoir	10,300	8,150	13,178	10.8	1,420
Brush Creek	10,300	7,470	11,588	6.9	800
North Crow Creek between Reservoir and Diversion	8,500	7,320	9,305	6.9	640
Middle Crow Creek	26,300	7,850	31.866	-	3,060
Above Granite Reservoir	17,800	8,000	22,132	10.8	2,390
Below Granite Reservoir but above Diversion	8,600	7,520	9,734	6.9	670
South Crow Creek	9,600	7,650	11,238	6.9	780
Total Crow Creek above Diversions	64,900	7,750	77,175	-	6,700
Crow Creek below Diversions	124,600	6,450	110,718	1.0	1,100

Note: 1 One-in-ten-year drought with 9.77 inches of precipitation.



RAW WATER DELIVERY TO CHEYENNE, DROUGHT CONDITIONS

FIGURE NO. 2.6-5

During drought conditions, historical records indicate a reduction in water demand. This is apparently due to the successful implementation of voluntary conservation measures. An assumption that water demand would remain at average values during drought conditions is therefore conservative.

2.6.1.1.6.2 Cheyenne Stage II Water Diversion Project

The CBPU is currently in the process of expanding the existing Stage I water diversion system. This project is scheduled to be completed in 1985. Improvements will consist of extending and expanding the Little Snake River collection and diversion facilities, increasing the capacity of the pipeline to transfer water from the western slope, expansion of the Hog Park Reservoir from 2,970 acre-ft to 22,650 acre-ft, and enlargement of the capacity of Rob Roy Reservoir from 8,895 acre-ft to about 35,400 acre-ft.

This work will increase the total potential yield of the system to 23,200 acre-ft/yr. However, the current Stage II project does not involve any major increase in the capacity of the Lake Owen to Crow Creek pipeline. The amount of water that can be diverted from Douglas Creek to Crow Creek is estimated at 12,400 acre-ft/yr by the CBPU, which is about 5,000 acre-ft more than the current potential yield. Lower yields have been estimated by others (Wyoming Department of Economic Planning and Development 1983). A schematic of the water resource system as it will exist after initial Stage II construction was shown in Figure 2.6-1.

When future water demands in the Cheyenne Urban Area justify construction of an additional pipeline from Lake Owen to the Crow Creek Basin, the Stage II diversion project would provide approximately 19,500 acre-ft of water per year (Banner 1983a). This is less than the developed yield of 23,200 acre-ft/yr because of the operational necessity of providing excess diversions from the western slope in order to assure that the North Platte River Basin receives adequate replacement water. In addition, following the completion of the current expansion of Hog Park and Rob Roy reservoirs, the CBPU will be required to provide replacement water to the North Platte Basin for reservoir evaporation losses (CBPU 1983).

2.6.1.1.6.3 Cheyenne Water Conservation Program

In April 1982 a "Proposed Water Conservation Plan" was presented to the Cheyenne City Council by the Cheyenne Water Conservation Advisory Group. This Plan contained recommendations regarding outdoor water use, metering, rate structure, water saving devices, and public education. A number of the recommendations from this Plan are currently being implemented. There are no specific goals for reducing water use associated with the Conservation Plan; however, at the very least this should result in a stabilization of per capita use at current levels. Because the program relies upon voluntary conservation there is no way of guaranteeing any particular level of reduction in use; this analysis therefore assumes that per capita use will remain constant in future years. Specific recommendations of the conservation plan are discussed below.

Outdoor water use, i.e., irrigation, accounts for nearly 30 percent of the total water use for Cheyenne. This amounts to over 4,000 acre-ft/yr of which about 75 percent is for residential uses. The Plan recommends implementing a program to inform the public of proper, i.e., efficient, watering methods and

soil and plant maintenance techniques and to encourage the use of plants and landscape designs that have low water requirements. Because this component of water use represents such a substantial portion of the area's water use the potential for savings is great. Benefits in this area would accrue gradually as the water users implement more efficient practices.

The Plan also recommended that all individual water users be metered in order to more fully account for water use. Currently, individual users in South Cheyenne and on F.E. Warren AFB are not metered nor charged on the basis of actual use. Until recently much of the civic water use was unmetered, including the irrigation use for parks and cemeteries. Also golf course irrigation use was metered but unbilled.

The Conservation Plan also recommended the implementation of a uniform rate structure and the billing of all water users. These two items may have the greatest overall impact of any of the recommendations in the Plan. The CBPU is currently in the process of establishing, over a 3-year period, a flat rate system in place of the existing declining rate system. As of August 1982, monthly water rates were 97 cents per 1,000 gallons for the first 250,000 gallons used; charges for additional use were 78 cents per 1,000 gallons. By July 1985, rates are projected to be $1.32 per 1,000 gallons for all levels of use. This not only constitutes a revision in the rate structure but also an increase of at least 36 percent over 1982 rates. This could provide substantial incentive for customers, particularly the larger users, to reduce any wasteful practices that may exist. In addition, billing of public water users will provide incentives to institute efficient water use practices, particularly for golf courses and park irrigation. The actual impact of the rate increase is difficult to estimate. However, in other areas the effect has often been that of decreasing use immediately followed by a gradual return to former levels of use as consumers become acclimated to the increased rates. Whether or not this will occur in the Cheyenne area is unknown.

The Plan also recommended that a program be established to encourage the use of low water use fixtures, water saving appliances, and efficient outdoor watering devices. The use of these items would be voluntary and would increase gradually with time as older items are replaced. The magnitude of the water savings cannot be estimated accurately.

Public education, also recommended by the Plan, will help in implementing the other recommendations of the Plan. Cheyenne has found that voluntary conservation is much more effective for reducing water demands than mandatory programs. The establishment of a public education program regarding water conservation would help to inform the public of the conservation alternatives available to them.

Successful implementation of water conservation programs can reduce overall water use by 5 to 10 percent, which in Cheyenne's case could reduce water demand by 700 to 1,400 acre-ft/yr. Because of the uncertainty of success and the relatively small incentive to conserve water (water costs are not a large portion of the typical users disposable income) no reduction in water demand by this measure has been included in existing plans of the CBPU.

2.6.1.2 Other Areas in the Region of Influence

In other ACSs, water is used primarily for agricultural irrigation; however, municipal, industrial, and private domestic water use is also important. This section summarizes all existing water uses in Platte, Goshen, Scotts Bluff, Banner, and Kimball counties, and a portion of Laramie County. Information is presented by county rather than watershed because much of the available data base is only available for the counties as a whole and potentially affected watersheds are in these counties. The water use is presented for the entire counties, with the exception of Laramie County, where data on the Crow Creek watershed are included in Section 2.6.1.1.

2.6.1.2.1 Water Use at Minuteman Facilities

Water is supplied by wells to LCFs at all Flights except C, where water quality and quantity problems require water to be trucked in. Wells exist at all LCFs. The wells were designed to deliver a minimum of 5 gallons per minute (gpm) and provide a daily supply of 1,500 gallons and to sustain a fire supply of 4,500 gallons. Storage is 5,500 gallons at each LCF (Porter and O'Brien 1962). Data on each well is given in Table 2.6-6. The water is treated, as necessary, to provide potable water. No increase in use is necessary for project operations. Most water used is discharged as wastewater. Current total Minuteman water use at LCFs is about 11.8 acre-ft/yr.

2.6.1.2.2 Municipal Water Use

There are currently 24 communities in the 6 counties included in other ACSs with municipal water systems. While all of these communities will not be impacted by the project they are included in this section for several reasons. Because of their proximity to the DA it is possible that some of these municipal supplies could potentially serve as sources of water for project construction. In addition, data for all communities are included for completeness and consistency with the other components of water use that are presented on a countywide basis. The 1980 water use and system capacities for all communities are presented in Table 2.6-7. Total municipal water use in 1980 was approximately 20,000 acre-ft. Groundwater is the source of all municipal water for these communities.

There are no long-term data available for the amount of water used by most of these communities, however, the data available show a widely varying per capita use throughout the area. Water use is generally high; some communities, most notably Torrington, provide water to industrial users which causes the per capita use to appear high. Data on these industrial water users are generally not available because many communities do not meter individual users. This may also contribute to the high rate of water use in the area because consumers pay a flat rate for their water regardless of the amount used.

2.6.1.2.3 Industrial Water Use

Industrial water users can be placed in two categories: those supplied by municipalities and those with their own supplies. The water use for municipally supplied industries is included in the figures reported in Table 2.6-7. The largest user in this category is Holly Sugar in Torrington, Wyoming.

Table 2.6-6

MINUTEMAN LAUNCH CONTROL FACILITY WELLS

	Construction Features			Pump Test		Quality (untreated)			Current Use
Flight	Depth of Well (feet)	Depth to Water (feet)	Formation[1]	gpm	Drawdown (feet)	TDS	NO_3-N (mg/l)	Iron	acre-ft/yr
A	282	165	AK	16.8	17	174	0.8	1.2	1.3
B	260	202	AK	16	19.5	244	1.1	3.8	1.3
C	870	132	LA	7.5	443	1,142	0	1.8	1.2
D	400	280	OG/WR	20	21	217	2.7	0.3	1.2
E	328	182	WR	20	4	232	3.1	0.3	1.1
P	150	78	AK	50	14	222	0.9	0.4	1.1
Q	152	89	WR	15	9	258	2.5	0.3	1.2
R	265	206	LA	15.2	13	282	0.4	0.1	1.2
S	282	49	LA	10	150	878	0	0.2	1.0
T	385	155	WR	31.4	27	330	1.2	0.2	1.2

Note: 1 AK = Arikaree, WR = White River, LA = Lance, OG = Ogallala.

Source: Porter and O'Brien 1962, F.E. Warren AFB 1983.

Table 2.6-7

MUNICIPAL WATER USE IN OTHER AREAS OF CONCENTRATED STUDY

Community	1980 Service Population	1980 Use, acre-ft/yr	1980 Use, gpcd	Current Raw Water Supply			
				Number of Wells	Peak[1] Capacity mgd	Rights[2] acre-ft/yr	Sustained[3] Capacity acre-ft/yr
Glendo	367	60	147	2	0.36	1,980	200
Hartville	149	62	369	3	0.45	340	250
Guernsey	1,512	334	197	3	1.22	2,500	685
Wheatland	5,816	2,480	380	12	6.78	10,390	3,800
Chugwater	282	52	165	4	0.28	290	155
Fort Laramie[4]	356	278	697	3	1.67	2,650	610
Lingle	475	150	282	3	1.44	2,540	805
Torrington[5]	6,100	4,820	705	12	15.09	15,970	8,450
Yoder	110	14	110	6	0.10	430	55
La Grange[6]	232	37	142	1	0.58	1,370	175
Albin	128	70	489	3	0.36	400	200
Burns	270	67	222	4	0.94	1,050	525
Pine Bluffs	1,077	513	425	7	4.90	8,025	2,745
Henry	155	46	264	2	1.08	1,210	605
Lyman	551	669	1,084	3	1.15	1,660	645
Morrill	1,097	612	498	4	4.25	5,240	2,380
Mitchell	1,956	538	245	4	3.60	5,320	2,015
Scottsbluff	15,026	4,410	262	14	19.70	26,550	11,035

Table 2.6-7 Continued, page 2 of 2
MUNICIPAL WATER USE IN OTHER
AREAS OF CONCENTRATED STUDY

Community	1980 Service Population	1980 Use, acre-ft/yr	1980 Use, gpcd	Current Raw Water Supply			
				Number of Wells	Peak[1] Capacity mgd	Rights[2] acre-ft/yr	Sustained[3] Capacity acre-ft/yr
Terrytown	727	189	233	4	6.9	4,840	3,465
Gering[7]	7,760	3,075	354	12	11.60	14,760	6,500
Minatare	969	287	264	3	1.95	2,900	1,090
Bushnell	187	55	264	2	1.08	1,610	605
Kimball[8]	3,170	1,000	282	13	3.15[a]	6,240	1,805[a]
Dix	279	246	788	3	1.12	1,210	625

Notes:
1 Assumes 24-hour/day operation of all usable wells.
2 Assumes 24-hour/day use of permitted wells.
3 Assumes 12-hour/day operation of all usable wells.
4 Peak capacity includes use of auxiliary fire pump.
5 Includes South Torrington Water and Sewer District.
6 Daily use limited by WSEO, new 450-gpm well permitted.
7 Three of the wells are on standby.
8 Seven of the wells are on standby or disconnected.
a Two wells included at reduced capacity.

Self-supplied industrial users are summarized by county in Table 2.6-8. Approximately 75 percent of industrial water use is from surface sources. The industrial use reported for Platte County is entirely for the Missouri Basin Power Plant near Wheatland, which is the single largest industrial user in the ROI. The Laramie County industrial use consists of oil production and secondary recovery use (Libra et al. 1981). Approximately 9,800 acre-ft of groundwater was used by the Scotts Bluff Power Plant in 1980 (Lawton et al. 1983). Both Banner and Kimball counties are large oil and gas producers. The actual water use for this purpose in each of these counties is not known, but the total use for the 2 counties is estimated at 1,200 acre-ft/yr based upon Nebraska well registration records.

Table 2.6-8

SELF-SUPPLIED INDUSTRIAL WATER USE
IN OTHER AREAS OF CONCENTRATED STUDY

County	Groundwater acre-ft/yr	Surface Water acre-ft/yr	Total acre-ft/yr
Goshen	Negligible	40	40
Laramie	230	Negligible	230
Platte	2,750	20,500	23,250
Banner	550	Negligible	550
Kimball	650	Negligible	650
Scotts Bluff	14,280[a]	5,060	19,340
TOTAL:	18,460	25,600	44,060

Note: a Does not include oil industry use which is small.

2.6.1.2.4 Commercial Water Use

Self-supplied commercial water users in other ACSs account for a very minor portion of the total water use. These include commercial establishments such as motels, restaurants, bars, etc. They account for an estimated 40 acre-ft/yr in Goshen County, 28 acre-ft/yr in Laramie County (outside of the Crow Creek watershed) and 44 acre-ft/yr in Platte County (Libra et al. 1981). No data are available for self-supplied commercial water use in Nebraska, but NDH records indicate that there are very few establishments which serve 25 or more people per day. It is assumed that the self-supplied commercial water use for the 3 counties in the Nebraska portion of the ROI is negligible.

2.6.1.2.5 Nonmunicipal Domestic Water Use

A relatively high percentage of the population in other ACSs relies upon nonmunicipal water sources. These include people residing in trailer courts, rural subdivisions, communities without central water systems, and those living on farms and ranches. This water use is summarized in Table 2.6-9. According to current NDH records there are six trailer courts in Scotts Bluff County and two in Kimball County with their own water systems. Total water use supplied by noncommunity water systems in these areas is estimated at about 260 acre-ft/yr.

About 23,000 people in the other ACSs rely upon individual wells. This represents one-third of the population of these areas. Water use from individual wells is estimated at 100 gpcd and totals approximately 2,530 acre-ft/yr.

2.6.1.2.6 Agricultural Water Use

2.6.1.2.6.1 Irrigation

The single largest water demand in other ACSs is for agricultural irrigation. Both ground and surface water are used throughout the area. The most heavily irrigated areas are along the North Platte River in Goshen and Scotts Bluff counties; irrigation in this area is primarily from surface water, while

Table 2.6-9

NONMUNICIPAL DOMESTIC WATER USE IN OTHER ACSs

County	Noncommunity		Individual Well[1]	
	Population	Use, acre-ft/yr	Population	Use, acre-ft/yr
Goshen	81	5	4,686	525
Laramie	51	4	2,892	324
Platte	230	79	3,619	405
Banner	0	0	918	103
Kimball	90	19	1,154	129
Scotts Bluff	857	56	9,300	1,042
TOTAL:	1,309	263	22,569	2,528

Note 1 Estimated at 100 gpcd

in the southern and western areas of the ROI groundwater is more heavily used. Crops grown with irrigation include wheat, barley, oats, corn, alfalfa, beans, and sugar beets. Corn, beans, and sugar beets are the primary crops irrigated with surface water. Table 2.6-10 provides a breakdown of irrigated acreages and water use by county for the other ACSs.

Table 2.6-10

IRRIGATION WATER USE IN OTHER ACSs

Irrigation	1981 Total Irrigation Acres	Irrigated With Groundwater, Acres	Groundwater Used, Acre-ft	Irrigated With Surface Water, Acres	Surface Water Used, Acre-ft	Total Water Used, Acre-ft
Wyoming						
Goshen	102,000	25,000	38,000	77,000	114,000	152,000
Laramie[1]	34,700	19,400	27,600	15,300	21,000	48,600
Platte	44,000	33,000	46,000	11,000	20,000	66,000
TOTAL:	180,700	77,400	111,600	103,300	155,000	266,600
Nebraska						
Banner	25,000	24,000	26,000	1,000	1,000	27,000
Kimball	30,000	29,000	31,000	1,000	1,000	32,000
Scotts Bluff	206,000	43,000	57,000	163,000	282,000	339,000
TOTAL:	261,000	96,000	114,000	165,000	284,000	398,000
ACS TOTAL:	441,700	173,400	225,600	268,300	439,000	664,600

Note: 1 Excludes Crow Creek Watershed

Total irrigation water use in other ACSs is about 665,000 acre-ft/yr. Of this amount, 60 percent is used in Nebraska. Of the total amount approximately one-third is supplied from groundwater sources.

The surface water use reported in Table 2.6-10 reflects the consumptive use by crops, rather than the actual water withdrawn for irrigation. Actual surface water withdrawals from the North Platte River are approximately 3 acre-ft per acre (Nebraska Natural Resources Commission 1983) with consumptive use of somewhat less than half that amount. The excess water withdrawn either goes to recharge the groundwater system or is returned to the North Platte through a series of drains that have been developed in the area. Irrigation use per acre varies throughout the ROI based upon variations in climate and water availability.

2.6.1.2.6.2 Livestock

Another agricultural use of water is for livestock watering. There are approximately 430,000 cattle and 79,000 sheep in other ACSs (Wyoming Crop and Livestock Reporting Service 1982, Nebraska Department of Economic Development 1983). Based upon water consumptions of 15 and 3 gallons per animal per day the annual use by livestock is approximately 7,500 acre-ft/yr. There are no accurate records to determine the sources of this water, but a recent report (Nebraska Natural Resources Commission 1983) indicates that for the North and South Platte River basins a breakdown of 80 percent groundwater and 20 percent surface water would be representative. Table 2.6-11 summarizes livestock water use in the ROI.

A summary of all existing water uses in other ACSs is presented in Table 2.6-12. As shown, irrigation accounts for about 90 percent of all water use, and nearly 63 percent of all water used is surface water.

Table 2.6-12

SUMMARY OF EXISTING (1980) WATER USE IN OTHER AREAS OF CONCENTRATED STUDY (acre-ft/yr)

	Groundwater	Surface Water	Total
Municipal	20,060	Negligible	20,060
Industrial/Commercial	18,460	25,600	44,060
Irrigation	225,600	439,000	664,600
Livestock	6,000	1,500	7,500
Domestic (nonmunicipal)	2,790	Negligible	2,790
TOTAL:	272,910	466,100	739,010

Table 2.6-11

LIVESTOCK WATER USE IN OTHER ACSs

	Cattle (Head)	Water Use (Acre-ft)	Sheep (Head)	Water Use (Acre-ft)	Total Water Use[1] (Acre-ft)
Wyoming	238,000	4,000	42,800	150	4,150
Goshen	108,000	1,810	14,000	50	1,860
Laramie[2]	45,000	760	21,800	80	840
Platte	85,000	1,430	7,000	20	1,450
Nebraska	192,000	3,230	36,000	120	3,350
Banner	24,000	405	1,000	5	410
Kimball	23,000	390	6,000	20	410
Scotts Bluff	145,000	2,435	29,000	95	2,530
ACS TOTAL:	430,000	7,230	78,800	270	7,500

Notes: 1 Assumed to be 80% groundwater, 20% surface water.

2 Excludes Crow Creek watershed.

2.6.2 Constraints On Water Use

2.6.2.1 Wyoming

2.6.2.1.1 Water Law

The right to use water in Wyoming is granted under the appropriation doctrine. The basic premise of the appropriation doctrine is "first in time is first in right," i.e., those who first perfect water rights will have priority over all those who later seek water rights in the same source.

In Wyoming, the doctrine of prior appropriation is constitutionally and statutorily codified into an administrative framework which sets forth the procedure for obtaining legally protected water rights and imposes certain legal constraints on the use of water.

2.6.2.1.1.1 Water Law Administration

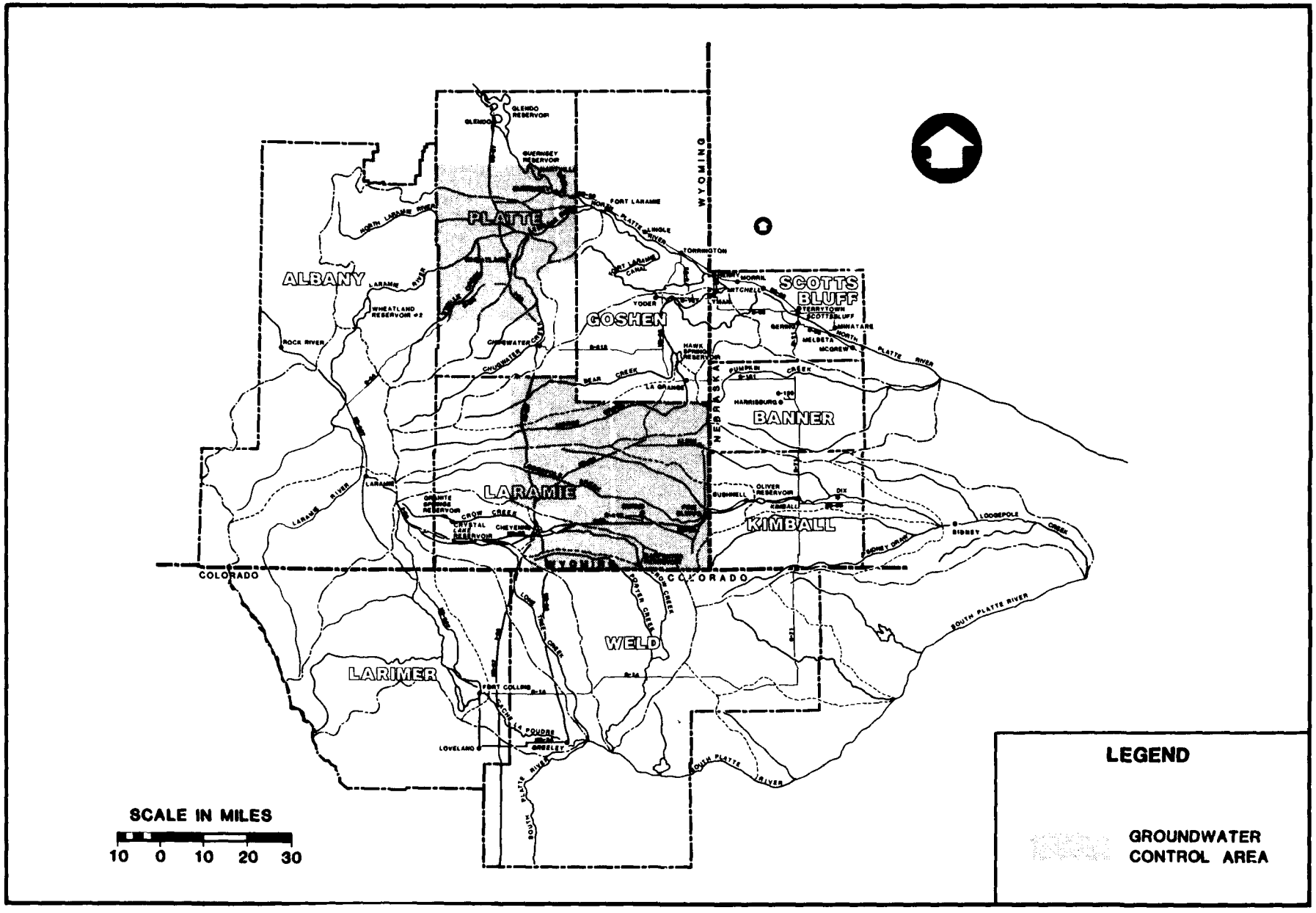
The four principal administrative organizations which confer and administer water rights are the State Engineer's Office, the Board of Control, Division of Advisory Committees, and the Control Area Advisory Boards. The State Engineer is appointed by the Governor and is empowered to issue permits to appropriate water and to issue rules, regulations, and orders concerning the use of water within the priority system. The Board of Control consists of the State Engineer and the superintendents of Wyoming's four water divisions. The Wyoming constitution vests in the Board of Control the power and duty to control and supervise the waters of the state. The Board is empowered to confer water rights through the issuance of certificates of appropriation and is also empowered to issue rules and regulations concerning the use of water.

In each of the four water divisions there is established an advisory committee consisting of three persons appointed by the Governor to represent landowners and water users in the division. The duties of the committees are to assist and advise the State Engineer and Board of Control regarding policies on the use and distribution of groundwater. Control Area Advisory Boards consist of five adult owners of land or groundwater rights within designated control areas. Members of advisory boards are elected by land and water rights owners within the control area and assist the other administrative organizations in the formulation of policies for groundwater use within the control area. Control areas have been established in Laramie County and in Platte County with boundaries shown on Figure 2.6-6.

2.6.2.1.1.2 Process for Obtaining Water Rights

Water rights can be obtained by perfecting a new water right, by purchase or lease of an existing perfected water right, or by obtaining a temporary permit for use of water.

The procedure for the perfection of a new water right depends on whether surface or groundwater is sought to be used. Generally, the use of any water without a permit is prohibited.



GROUNDWATER CONTROL AREAS

FIGURE NO. 2.6-6

Surface Water Rights. The filing with the State Engineer of a permit application setting forth the source, amount, and place of use is the first step in acquiring a perfected right to the use of surface water. The State Engineer must approve all such applications in proper form which contemplate the application of water to a beneficial use and where the proposed use neither tends to impair the value of existing rights nor appears to be otherwise detrimental to the public welfare. If there is no unappropriated water available in the proposed source of supply or if the proposed use conflicts with existing rights or threatens to prove detrimental to the public interest, the State Engineer must reject the application and refuse to issue the permit. Alternatively, to avoid injury to other water rights, the State Engineer may issue the permit with conditions, i.e., the total amount of water to be diverted and/or the period of time during which the water may be diverted. In addition, before passing an application, the State Engineer may require such additional information as would enable him to properly guard the public interest.

If the application is approved and the permit is issued, the permittee must commence and complete construction of the diversion structure within the time limits set in the permit. Generally, completion must be accomplished within 5 years of the issuance of the permit and in the case of a ditch permit, final proof of the appropriation on the requisite form must be submitted to the division superintendent or State Engineer within 5 years of completion of such construction. Default by the permittee on any of these specified requirements may, upon notice given by the State Engineer, work a forfeiture of the water right involved. An unadjudicated water right perfected pursuant to the permit receives the date of the application as the priority date.

If the appropriator perfects the unadjudicated water right pursuant to the permit, he may prepare a proof of appropriation and commence procedures to have his water right adjudicated. After final proof of appropriation has been submitted and after the appropriation has been verified by the proper division superintendent from a field inspection, the superintendent then gives public notice of the proof by advertisement in a newspaper of general circulation in the area. Should other affected water rights holders wish to protest the proposed adjudicated water right, the superintendent must hold a public hearing upon prior notice during which evidence is taken regarding the possible injurious effects of the proposed water right.

Upon completion of the taking of the final proof, the superintendent submits the final proof and any other evidence gathered if a hearing has been held to the Board of Control. At its next regular meeting the Board then considers the proof and other evidence. If the Board is satisfied that there are no conflicts with other water rights and that the appropriation involved has been perfected in accordance with the permit issued by the State Engineer, then the Board issues a certificate of appropriation. This certificate sets forth the name and address of the appropriator, priority date and number of the appropriation, the amount and use of the water appropriated and, if the water is to be used for irrigation, the legal description of the land to which the water is to be applied.

In addition, the certificate of appropriation may set forth certain conditions relating to the time of year during which the water may be used and/or maximum cumulative amount of water which may be diverted in any given year. The

priority date of the adjudicated right will be the date the application was filed with the State Engineer.

Groundwater Rights. To acquire a perfected water right for groundwater in a noncontrol area, the procedure is similar to the one set forth above regarding surface water rights. Proceedings are initiated by filing an application with the State Engineer for a permit setting forth the nature of the proposed use, location of the proposed well, information regarding local hydrological conditions, type or means of diversion and conveyance, amount of water requested and, if the water is to be used for irrigation, description of the land upon which the water is to be used. Such applications for wells in noncontrol areas are generally granted as a matter of course if the proposed use is beneficial and if the State Engineer finds that the proposed means of diversion and construction are adequate.

If, however, the State Engineer finds that approving the application would not be in the public interest regarding water use, e.g., if no unappropriated water is available or if existing water rights would be impaired, he may deny the application subject to review by the Board of Control. Once again the State Engineer may issue the permit but with conditions as to the total amount of water to be diverted and/or the period of time during which the water may be diverted.

If the permit is issued, the permittee must commence construction of diversion works within 1 year from the date of permit approval and must effect completion thereof and apply the water to beneficial use within the time specified in the permit, generally within 3 years. Upon completion and application to the beneficial use, the permittee must submit to the State Engineer detailed information regarding the location of the well and the means and extent of diversion of water. Once this information and actual application of the water to a beneficial use have been verified by a filed inspection, the Board of Control may consider the groundwater right for adjudication in the same manner prescribed for the adjudication of surface appropriations.

Additional procedures are mandated where a groundwater right is sought in a control area. Any underground water district or subdistrict may be designated a control area by the Board of Control when necessary to provide additional regulation of especially scarce groundwater resources. Such designation may be made for any of the following reasons: the use of underground water is approaching a use equal to the current recharge rate; groundwater levels are declining or have declined excessively; conflicts between users are occurring or are foreseeable; the waste of water is occurring or may occur; or other conditions exist or may arise that require regulation for the protection of the public interest.

Upon the filing of an application to appropriate groundwater from a control area, the State Engineer publishes in a newspaper of general circulation in the relevant county notice of the application and solicitation of objections to the granting of the application. If objections are filed, or if the State Engineer is of the opinion that the granting of the application may be detrimental to the public interest or if he wishes to obtain the recommendations of the Control Area Advisory Board, the State Engineer and the Control Area Advisory Board conduct a hearing upon prior notice to the applicant and any objectors. The application will be granted and the permit issued only after

the State Engineer finds, after receiving the advice of the Control Area Advisory Board, that there is unappropriated water in the proposed source, that the proposed means of diversion or construction are adequate, that the location of the proposed well does not conflict with any well spacing or distribution regulation, and that the proposed use would not be detrimental to the public interest. In any event, the permit may contain such conditions as the State Engineer may deem necessary for the protection of other water rights.

After the permit has been issued, the same time requirements and adjudication procedures which apply to groundwater rights in noncontrol areas are followed. Similarly, for groundwater rights in control areas, the priority shall date from the filing of the application with the State Engineer.

Temporary Water Rights. In addition to, or in conjunction with, obtaining a new water right, any person may acquire by purchase, lease, or gift the right to the temporary use of water which may be embraced in any adjudicated or valid unadjudicated water right or any portion thereof. The water may be used under its original priority date for a period not exceeding 2 years.

Before any right to such temporary use shall become operative, a written application for approval must be made to the State Engineer. Such applications are approved only if no other appropriator will be injured thereby. At the end of the temporary use period the water right will automatically revert to the original appropriator.

Existing Rights and Water Exchanges. In Wyoming a water right is a legally recognized property right and may be purchased or leased from an existing owner. This generally involves negotiating a water right deed and submitting for State Board of Control approval if a change in place or type of diversion or use is proposed.

A water right may and usually does exist apart from the land upon which it is found. Further, a water right may be held or conveyed separately from the land upon which it is used or found. However, a Wyoming water right is a property right that confers only the right to use water and not the right to exercise complete dominion and control over water. Indeed, by statute, all water belongs to the State and the right to its use must be acquired in the manner provided for by the State in its statutes and the regulations promulgated thereunder.

By a recent statutory enactment Wyoming now permits the owner of an existing water right to change its type or place of use. Thus, an industrial user may, for example, purchase an existing agricultural water right and change its place and type of use in order to augment water supply. However, such changes require the approval of the State Engineer and are subject to a variety of conditions and limitations.

Water exchanges are also possible in Wyoming. Any appropriator owning a valid water right to either surface, ground, or reservoir water may petition the State Engineer for an order allowing the exchange among water right holders of any combination of water from various physical sources. The State Engineer may not issue an order approving such an exchange if it would adversely affect other appropriators or would be too difficult to administer. Accordingly, any

approval order may include conditions to avoid injury to other water users. Termination of an exchange also requires an order from the State Engineer.

2.6.2.1.1.3 Constraints on Water Use

Beneficial Use. In Wyoming, as in other appropriation states, beneficial use is at all times the basis, measure, and limit of the right to use water.

In order to discourage waste and encourage optimum water utilization, a water right exists in terms of use and is limited to only that amount of water which, using reasonably efficient means of diversion and application, can be beneficially used. Thus, although the amount of water permitted or adjudicated to a water right is prima facie evidence of the entitlement thereunder, this may be rebutted by evidence of actual beneficial historic use. This concept is further recognized by statutory procedures wherein all or part of a water right may be declared abandoned through nonuse.

Beneficial use is related to a statutory provision setting a maximum "duty of water" in irrigation. In Wyoming no allotment for the direct use of the natural unstored flow of any stream may exceed 1 cfs for each 70 acres of land for which the appropriation is made. (Note: this limitation does not apply to early water rights adjudicated by Wyoming territorial courts.)

Further, water rights are "appurtenant," i.e., they generally cannot be detached from the lands, place, or purpose for which they are acquired without permission of the State. For example, agricultural water rights may not simply be purchased and immediately used at another place for another use.

Change in Type or Place of Use. An owner of a water right who wishes to change the type or place of use thereof must petition the Board of Control for permission to make the change. Such petition must set forth all pertinent facts relating to both the present and proposed type and place of use and the Board may require that an advertised public hearing be held on the petition.

The Board may approve the change provided that the quantity of the water so transferred does not exceed the amount of water historically diverted under the existing use, nor decrease the historic amount of return flow, nor in any manner injure other existing water rights. In addition, the Board must consider all other facts relevant to the proposed change including the economic loss to the community and state of the use from which the water is transferred, the extent to which such economic loss is offset by the new use, and whether other sources of water are available for the new use.

Recent Wyoming Supreme Court decisions have rejected the rule announced in early cases to the effect that any change in a water right is permitted if no injury results to other water users. For example, a change which increases historic consumptive use would not be permitted even if no other appropriator is injured thereby.

Notably, changes in water rights are not limited by purely legal considerations. The water right change statute permits the Board to engage in a value judgment as to whether the proposed change would generally benefit the state. For example, it is possible that the Board could deny a change from agricultural to industrial use if it felt the loss from the discontinued use outweighed any benefits derived from the new use.

The facts of a recent Wyoming Supreme Court case provide a good example of the operation of the water right change statute. In that case, a municipality purchased an agricultural groundwater right and sought to change it to a new location and to municipal use. The well right had a priority date of 1920 and was for 1,100 gpm and was used to irrigate 39 acres. Yet, over the range of its lifetime the well had on average produced 138 acre-ft annually to irrigate 110 acres. The Board allowed both changes but with 2 limitations affirmed by the Court, viz, that the water would be withdrawn only during the period from May 15 to October 15 of each year at a rate of withdrawal not to exceed 1,100 gpm until such time as a total withdrawal of 49 acre-ft was reached. This latter figure was the amount of water historically consumptively used under the original water right and was the maximum amount available for transfer under the statute. Further, because the discontinued use took place only during the historic irrigation season, the time of diversion for the changed use could not be expanded without injury to other water users.

If the municipality wished to use water during the nonirrigation season, it would either have to purchase and receive approval for a change in a right not historically used during the irrigation season or store the changed irrigation right during the irrigation season for later use.

This case points out an important constraint in water rights and changes therein. Because of basic principles of water law and statutory refinements thereof, any change in a water right will be significantly limited in amount and time of use. Moreover, care must be taken to verify recent historic beneficial use and consumptive use to make certain a water right for acquisition and change is not susceptible to objections based on abandonment or waste (i.e., nonbeneficial use).

These same difficulties are present when an exchange or temporary water right is sought. An exchange is permitted only if other appropriators are not injured. All exchanges are subject to the requirements of beneficial use and equality of water exchanged. No exchange will be allowed unless a sufficient quantity of makeup, i.e., replacement water is introduced to replace the water diverted and withdrawn under the exchange as determined by the relative consumptive uses and transmission losses. In the case of a temporary water right, only that portion of the water right so acquired which has been historically consumptively used may be diverted by a temporary user. At least in the instance of temporary surface right the amount of historic consumptive use is presumed to be 50 percent. Accordingly, the presumed historic return flow of 50 percent must be left in the stream for use by downstream appropriators.

Priority Based on Type of Beneficial Use. In addition to priority based upon time of appropriation and application to beneficial use, Wyoming features a preference based upon type of use. Types of use are classified as either preferred or nonpreferred. Existing rights not preferred may upon approval by the Board of Control be condemned to supply water for preferred uses in

accordance with provisions of the law relating to condemnation of property for public and semipublic purposes. As is required in any condemnation, just compensation must be provided for any water taken.

Preferred water uses have preference in the following order: drinking water and stock water; municipal; steam engines, general railway use, culinary, laundry, bathing, refrigeration, steam, and hot water heating plants and steam power plants; and industrial. The use of water for irrigation is superior and preferred to any use where water turbines or impulse waterwheels are installed for power purposes; however, the preferred use of steam power plants and industrial purposes are not construed to give the right of condemnation. All uses, whether preferred or not, are regulated within the temporal priority system.

In addition to these general use preferences, Wyoming confers special preference upon certain small domestic wells. Appropriations of groundwater for domestic and stockwater purposes (including a maximum of 1 acre of irrigated lawns and gardens) where the yield or flow does not exceed 25 gpm have preferred right over all other rights regardless of their priority dates. If another nonpreferred well unreasonably interferes with such a preferred domestic or stockwater well the State Engineer may, upon complaint of the operator of the latter, order the interfering appropriator to cease or reduce withdrawals or to compensate the injured operator in kind. As between two or more such preferred wells, that appropriation having the earliest priority date shall have the preferred right.

Interstate Requirements. Rights to the use of water in interstate rivers and streams are generally regulated by interstate compact ratified by Congress or by a decree from the United States Supreme Court equitably apportioning the available waters among the affected states. For example, the 1945 United States Supreme Court case of Nebraska vs. Wyoming apportions the water of the North Platte River between those two states based upon principles similar to the appropriation doctrine. This case could impact changes that may occur in Horse Creek which drains to the North Platte River. The rules, agreements, and orders found in the various compacts and Supreme Court cases are binding upon all states involved and generally constitute an additional legal constraint upon the use of water within each state. No legal requirements exist for flow of Wyoming streams that drain to the South Platte River in the ROI (i.e., Crow and Lodgepole creeks).

Water Rights Versus Water Use. Another important constraint on the use of water within the State of Wyoming is the great deal of uncertainty surrounding all water rights arising from the discrepancy between the information contained in water rights permits or certificates and actual use.

Early adjudications by the Board of Control have resulted in inaccuracies in amount and place of use and in a proliferation of "paper water rights," many of which have fallen into disuse or were never actually used. Many permits issued years ago have never been perfected by construction of diversion structures and actual application of water to a beneficial use.

Forfeiture of such water rights does not occur automatically but rather in proceedings initiated by other water users. Moreover, some Wyoming court cases have indicated that resumption, or in some instances even initiation, of

appropriation may be a defense in such proceedings. There is even some doubt as to whether the State Engineer has standing to bring proceedings to forfeit such paper water rights. In addition, unrecorded and unapproved changes in points of diversion and places of use are widespread in Wyoming as is the use of water contrary to the limitation of 1 cfs per 70 acres.

In the state records of water rights tabulations, the sum of all water rights exceeds, often by a great amount, the average streamflow. In actual use it is usually the case that most such junior rights will be in priority only in years of high precipitation or only during that time of year when the streamflow is greatest, i.e., during spring runoff. In addition, because most water uses are only partially consumptive and result in substantial return flow, the total amount of water rights in a given stream are often greater than the streamflow at any given point.

Adjudicated surface water rights from state records (State of Wyoming 1981) have been summarized for three watersheds in Laramie County in Table 2.6-13. These rights greatly exceed flow available in the creeks. Similarly, tabulations of groundwater rights in terms of gallons per minute (gpm) of discharge have been made (Ertec Western, Inc. 1983b) and are shown in Table 2.6-14.

The possibility that these paper water rights with early priority dates may be resurrected for use places a cloud of uncertainty over all current or new water rights. They adversely affect the feasibility of all water projects, whether large or small, by making it difficult or impossible to determine the availability of unappropriated water or the actual value of any given water right.

2.6.2.1.2 Water Quality

In addition to meeting certain legal requirements when obtaining water, there are also restrictions on returning the water to streams or groundwater. Waters in Wyoming have been classified according to their quality and any discharges to these waters must meet effluent standards so that the quality of the water is maintained.

2.6.2.1.2.1 Surface Water

There are four classes of surface water in Wyoming: Class I nondegradation streams; Class II presently supporting or capable of supporting game fish; Class III presently supporting or capable of supporting nongame fish; and Class IV not capable of supporting fish. In addition, all streams are also designated as "coldwater" or "warmwater" fisheries by the Wyoming Game and Fish Department. A summary of Wyoming Water Quality Standards appears in Table 2.6-15.

The classifications for the streams of interest in the ROI are given in Table 2.6-16. As seen in the table, none of the creeks are classified as being Class I streams. Water quality standards prevent new point sources from discharging to Class I streams. There are also no water quality limited segments in the ROI. Dischargers into such segments are required to provide ammonia removal in addition to secondary treatment to maintain the designated water quality. For those discharging to segments not defined as being water quality limited, secondary treatment is usually all that is necessary,

Table 2.6-13

ADJUDICATED SURFACE WATER RIGHTS IN LARAMIE COUNTY

	Crow		Lodgepole		Horse	
	cfs	acre-ft/yr	cfs	acre-ft/yr	cfs	acre-ft/yr
Irrigation	76.79	1,171.875	224.81	2,469.19	1,326.03	2,303.33
Domestic	17.2213	364.45	20.75	22.6	42.433	150.37
Fish	-	6.72	14.75	105.66	-	50.43
Flood Control	-	-	-	204.05	-	-
Stock	22.718	1,105.885	67.04	214.83	49.75	138.7
Recreation	-	47.98	-	17.06	-	-
Unknown	425.73	203.75	-	40.00	-	-
Municipal	30.08	10,985	-	-	-	-
Railroad	1.17	298.23	-	-	-	-
Fire	-	-	-	-	0.33	-
Manufacturing	-	-	-	-	0.33	-
Supply Ditch	-	-	-	-	1.36	-

Note: The rights for Crow Creek are those recorded as being upstream of the Colorado border, for Lodgepole Creek recorded as being upstream of the Nebraska border, and for Horse Creek recorded as being upstream of the Goshen County line. Those numbers given in acre-ft/yr should not be interpreted as corresponding to the rights in cfs, but are separate rights recorded by the State Engineer for April 1981.

Table 2.6-14

GROUNDWATER RIGHTS IN LARAMIE COUNTY
(gpm)

	Township					
Range	T18N	T17N	T16N	T15N	T14N	T13N
R69W	378	130	2,373	6,461	1,830	620
R68W	1,134	243	170	220	14,552	8,574
R67W	3,262	212	2,842	2,170	9,849	7,398
R66W	4,803	82	4,113	107	22,767	5,699
R65W	309	38	5,785	2,808	5,275	1,123
R64W	299	184	1,891	2,724	5,652	4,560
R63W	3,038	447	8,192	14,786	8,581	4,776
R62W	4,006	7,291	5,097	10,716	17,577	4,659
R61W	111	6,054	19,292	3,290	21,164	13,260
R60W	818	15,149	10,511	20,724	126,604	14,824

Note: The total rights (gpm) are 471,000 gpm which converts to 761,000 acre-ft/yr if pumping were continuous. Actual pumping is not continuous, of course, and has been limited to annual withdrawals in some cases. Irrigation well systems are designed for 800-1,000 gpm to irrigate 160 acres. On this basis (assuming 87 percent of rights are for irrigation) sufficient rights would exist to irrigate about 73,000 acres. Actual irrigation in 1980 from groundwater was about 26,000 acres in Laramie County.

Source: Based on data from Ertec Western, Inc., 1983.

Table 2.6-15

WYOMING WATER QUALITY STANDARDS
SUMMARY

Constituent	Class I	Class II	Class III	Class IV
Ammonia	0.02 mg/l	0.02 mg/l	0.02 mg/l	
Benzedine	0.0001 mg/l	0.0001 mg/l	0.0001 mg/l	
Chlorine				
Coldwater Fishery	0.002 mg/l	0.002 mg/l		
Coldwater Fishery	0.01 mg/l	0.01 mg/l	0.01 mg/l	
t-Radium 226+ Radium 228	5 pCi/l	5 pCi/l	5 pCi/l	5 pCi/l
T-Strontium	8 pCi/l	8 pCi/l	8 pCi/l	8 pCi/l
Turbidity	10 NTU Increase	10 NTU Increase	15 NTU Increase	
Dissolved Oxygen	>6 mg/l	>6 mg/l	>5 mg/l	
Temperature				
Coldwater Fishery	2°F Change	2°F Change	2°F Change	
Warmwater Fishery	4°F Change	4°F Change	4°F Change	
Coldwater Fishery	78°F Max.	78°F Max.	78°F Max.	
Warmwater Fishery	90°F Max.	90°F Max.	90°F Max.	
pH	6.5-9.0	6.5-9.0	6.5-9.0	6.5-9.0
Fecal Coliform				
Entire Year				Mean 200/ 100 ml, 10% not to exceed 400/ 100 ml in 30-day period

Table 2.6-15 **Continued**, page 2 of 3
WYOMING WATER QUALITY STANDARDS
SUMMARY

Constituent	Class I	Class II	Class III	Class IV
Fecal Coliform				
Full Body Contact[1] May 1 - Sept. 30	Mean 200/ 100 ml, 10% not to exceed 400/ 100 ml in 30-day period	Mean 200/ 100 ml, 10% not to exceed 400/ 100 ml in 30-day period	Mean 200/ 100 ml, 10% not to exceed 400/ 100 ml in 30-day period	
Secondary Body Contact[2]	Mean 1,000/ 100 ml, 10% not to exceed 2,000/ 100 ml in 30-day period	Mean 1,000/ 100 ml, 10% not to exceed 2,000/ 100 ml in 30-day period	Mean 1,000/ 100 ml, 10% not to exceed 2,000/ 100 ml in 30-day period	
Oil and Grease	10 mg/l	10 mg/l	10 mg/l	10 mg/l
Total Dissolved Gases Below Manmade Dams	110%	110%	110%	110%

Notes: 1 Class I, II, and III waters identified as being suitable for full body contact recreation:
- a All still surface water bodies which lie at an elevation of less than 7,000 feet above sea level.
- b The North Platte River from the City of Casper's water supply intake upstream to the Wyoming-Colorado state line.
- c The Snake River through its entire length in Wyoming.
- d The Green River through its entire length in Wyoming.
- e The main stem of the Tongue River through its entire length in Wyoming.
- f The Encampment River through its entire length in Wyoming.
- g The Big Horn River from the Wyoming Highway 172 bridge upstream to the wedding of the waters.
- h Goose Creek through its entire length.

Table 2.6-15 Continued, page 3 of 3
WYOMING WATER QUALITY STANDARDS SUMMARY

i Little Goose Creek through its entire length.
j Clear Creek at the point of intake to the town of Buffalo's swimming pool.
k Flat Creek (near Jackson, Wyoming) through its entire length.
l Fish Creek (near Wilson, Wyoming) through its entire length.
m The Hoback River through its entire length.

2 All surface waters not designated as Class IV or described in Note 1 are considered suitable for secondary body contact recreation.

Source: Wyoming Department of Environmental Quality (WDEQ) 1981.

Table 2.6-16

CLASSIFICATION OF STREAMS OF INTEREST IN WYOMING

Class I	Class II	Class III	Class IV
None	North Platte River	Muddy Creek	North Laramie River
	Laramie River	Lodgepole Creek below U.S. 85	Bear Creek
	Chugwater Creek		Chivington Draw
	Sybille Creek		Crow Creek below Cheyenne
	Horse Creek		Most Canals and Irrigation Diversions
	Crow Creek above Cheyenne		
	Lodgepole Creek above U.S. 85		

although the discharger is responsible for seeing that water quality standards for substances such as toxics, oil and grease, and dissolved oxygen are not violated. Secondary treatment effluent standards are given in Table 2.6-17.

Before a point source discharge is allowed, a permit must be issued by the WDEQ. Such a permit will outline the effluent limitations to be met by the discharge in question so that the stream quality is protected. If it is found that a stream segment is being polluted by nonpoint sources, best management practices will be implemented in the area.

2.6.2.1.2.2 Groundwater

Groundwater in Wyoming is separated into seven classes. Water quality standards for the first four, Class I domestic use, Class II agricultural use, Class III livestock, and Class Special (a) are listed in Table 2.6-18. In addition to those standards, the above classes shall not contain biological, hazardous, toxic, or potentially toxic substances at concentrations which exceed maximum allowable concentrations.

Class IV is groundwater suitable for industrial use and has two subclasses. Class IV (A) has total dissolved solid (TDS) concentrations not exceeding 10,000 milligrams per liter (mg/l) and Class IV (B) has TDS concentrations in excess of 10,000 mg/l. Further standards have been established for industrial use because of the highly variable requirements of different industries.

Class V groundwaters are those found closely associated with commercial deposits of hydrocarbons and/or other minerals, or which are considered a geothermal resource. Class V consists of Class V (hydrocarbon commercial), Class V (mineral commercial), and Class V (geothermal).

Class VI waters are those that are considered unusable. This could be due to poor water quality (high TDS, highly contaminated), or other factors.

Groundwater in Wyoming is not classified until:

- There is pollution or the threat of pollution to the groundwater of the state.
- The physical, chemical, radiological, or biological properties of any groundwater of the state are or may be altered by man's action (WDEQ 1980).

The groundwater in the ROI has not yet been classified by the WDEQ. When a groundwater supply has not been classified, it still must be considered as being classified according to its use or potential use. That is, if the water is of high enough quality to be used for domestic purposes, it must then be considered as being classified as a Class I groundwater (WDEQ 1983).

Discharges to groundwater must be such that they do not make the affected water unsuitable for its intended use or uses, at any place or places of withdrawal or natural flow to the surface.

Table 2.6-17

SECONDARY TREATMENT EFFLUENT STANDARDS

	Lagoons	Mechanical Plants
TSS	100 mg/l	30 mg/l
BOD_5	30 mg/l	30 mg/l
pH	6.5-9.0	6.5-9.0

	Contact Recreation	Noncontact Recreation	Class IV Streams
Fecal Coliform	200/100 ml	1,000/100 ml	200/100 ml

	Class II & III Streams	Class IV Streams
Total Residual Chlorine:	0.5 mg/l	No Cl Limit

Source: WDEQ 1982.

Table 2.6-18

UNDERGROUND WATER QUALITY STANDARDS

Use Suitability Constituent or Parameter	Class I Domestic Concentration[1]	Class II Agriculture Concentration[1]	Class III Livestock Concentration[1]	Special (A) Fish/Aquatic Life Concentration[1]
Aluminum (Al)	-	5.0	5.0	0.1
Ammonia (NH_4)	0.5	-	-	0.016
Arsenic (As)	0.05	0.1	0.2	0.05
Barium (Ba)	1.0	-	-	5.0
Beryllium (Be)[b]	-	0.1	-	0.011-1.1
Boron (B)	0.75	0.75	5.0	-
Cadmium (Cd)[b]	0.01	0.01	0.05	0.0004-0.015
Chloride (Cl)	250.0	100.0	2,000.0	-
Chromium (Cr)	0.05	0.1	0.05	0.05
Cobalt (Co)	-	0.05	1.0	-
Copper (Cu)[b]	1.0	0.2	0.5	0.01-0.04
Cyanide (CN)	0.2	-	-	0.005
Fluoride (F)[a]	1.4-2.4	-	-	-
Hydrogen Sulfide (H_2S)	0.05	-	-	0.002
Iron (Fe)	0.3	5.0	-	0.5
Lead (Pb)[b]	0.05	5.0	0.1	0.004-0.15
Lithium (Li)	-	2.5	-	-
Manganese (Mn)	0.05	0.2	-	1.0
Mercury (Hg)	0.002	-	0.00005	0.00005
Nickel (Ni)[b]	-	0.2	-	0.05-0.4
Nitrate (NO_3 -N)	10.0	-	-	-

Table 2.6-18 Continued, page 2 of 3
UNDERGROUND WATER QUALITY STANDARDS

Use Suitability Constituent or Parameter	Class I Domestic Concentration[1]	Class II Agriculture Concentration[1]	Class III Livestock Concentration[1]	Special (A) Fish/Aquatic Life Concentration[1]
Nitrite (NO_2 -N)	1.0	-	10.0	-
Nitrate + Nitrite (NO_3 + NO_2 -N)	-	-	100.0	-
Oil & Grease	Virtually Free	10.0	10.0	Virtually Free
Phenol	0.001	-	-	0.001
Selenium (Se)	0.01	0.02	0.05	0.05
Silver (Ag)[b]	0.05	-	-	0.0001-0.00025
Sulfate (SO_4)	250.0	200.0	3,000.0	-
Total Dissolved Solids (TDS)[c]	500.0	2,000.0	5,000.0	500.0-1,000.0-2,000.0
Uranium (U)[b]	5.0	5.0	5.0	0.03-1.4
Vanadium (V)	-	0.1	0.1	-
Zinc (Zn)[b]	5.0	2.0	25.0	0.05-0.6
pH	6.5-9.0	4.5-9.0	6.5-8.5	6.5-9.0
SAR	-	8	-	
RSC	-	1.25 meq/L	-	
Combined Total Radium 226 and Radium 228	5pCi/L	5pCi/L	5pCi/L	5pCi/L
Total Strontium 90	8pCi/L	8pCi/L	8pCi/L	8pCi/L
Gross alpha particle activity (including Radium 226 but excluding Radon and Uranium)	15pCi/L	15pCi/L	15pCi/L	15pCi/L

Table 2.6-18 Continued, page 3 of 3
UNDERGROUND WATER QUALITY STANDARDS

Notes: 1 mg/l, unless otherwise indicated

a Dependent on the annual average of the maximum daily air temperature: 1.4 mg/l corresponds with a temperature range of 26.3 to 32.5 degrees C and 2.4 mg/l corresponds with a temperature of 12.0 degrees C (53.7 degrees F) and below.

b Dependent on hardness: The toxicity of metals in natural waters varies with the hardness of the water; generally, the limiting concentration is greater in hard water than in soft water.

c Total dissolved solids limits are 500 mg/l for egg hatching, 1,000 mg/l for fish rearing, and 2,000 mg/l for general fish and aquatic life.

Source: WDEQ 1980.

2.6.2.2 Nebraska

2.6.2.2.1 Water Law

Nebraska operates under a mixed riparian-appropriative system of water law. Prior to April 4, 1895, court decisions established that surface water would be allocated on the basis of riparian rights. A riparian water right is the right of an owner of land that abuts a natural stream to make beneficial use of water from the stream. On April 4, 1895, Nebraska adopted statutes that created a system of appropriative water rights. An appropriative water right is a right to appropriate (divert from a natural stream) and make beneficial use of surface water. Subsequent to April 4, 1895, many owners of a riparian right secured appropriative water rights, and certain lands with a riparian right have been severed from the stream and sold to new owners. In these instances the riparian right is lost. However, some riparian rights that existed prior to 1895 are still valid and when conflicts occur the courts still recognize riparian rights.

On August 23, 1975, the Nebraska Groundwater Management Act became effective. Its purpose was to provide for orderly management practices to conserve groundwater for beneficial use. The Act established procedures for designating groundwater control areas in the state and it mandated a permit system for the use of groundwater in control areas for all purposes except domestic uses. In noncontrol areas, permits are required only for industrial uses that exceed 3,000 acre-ft/yr, for violation of well spacing, and for using water on adjoining lands in other states (Ertec Western, Inc. 1983a).

2.6.2.2.1.1 Water Law Administration

The Department of Water Resources has jurisdiction over all matters pertaining to water rights for surface water irrigation, power, or other beneficial purposes. The Department may adopt rules and regulations governing the appropriation and beneficial use of water, and it may refuse to allow any water to be used by claimants until their rights have been determined and made of record. It may request information relative to water use from county, irrigation, power, or drainage offices, and from any other person or persons, and it may hold public hearings on complaints, petitions, or applications concerning water use. During public hearings, the Department will receive evidence relevant to the matter under investigation and the burden of proof will be rested on the person making the complaint, petition, or application. Following each hearing the Department will issue appropriate orders, in writing, concerning the contested issue.

Anyone dissatisfied with a decision of the Department may appeal to the Supreme Court of Nebraska to reverse or modify the order. The appeal must be made within 1 month from the issuance of the order. In rendering its decision the appellate court will consider only the evidence presented to the Department and a transcript of the record on which the decision was based.

For the purpose of administering and regulating the use of the available water supply, the state is divided into two water divisions. Water Division No. 1 consists of all lands of the state contained within the Platte rivers and their tributaries lying west of the mouth of the Loup River, and all other lands lying south of the Platte and South Platte rivers. Water Division No. 2

consists of all land within the Loup, White, Niobrara, and Elkhorn rivers and their tributaries, and all other lands of the state not included in Water Division No. 1.

Each water division is subdivided into six water districts which conform to major watershed boundaries. A division engineer is the chief administrator of each division and he directs the work of local water commissioners. There is at least one water commissioner in each water district. During times of water deficiencies it is the responsibility of the division engineer and water commissioners to regulate the water resources in accordance with state law, under the general direction of the water resources director.

2.6.2.2.1.2 Process for Obtaining Water Rights

Rights to the beneficial use of water in Nebraska may be acquired by surface water appropriation, by purchasing land with an appropriative water right of record, or by owning land overlying a groundwater reservoir. In general, surface water may not be used without a valid water-right permit, permits are required for groundwater use only in control areas for all uses except domestic, and in noncontrol areas for certain industrial and municipal uses. Appropriation procedures and permit requirements will be described in the following sections.

Surface Water Rights. The water of every natural stream in Nebraska not currently appropriated is considered to be the property of the public, is dedicated to the people of the state, and is subject to appropriation through a permit system. The priority of appropriation dates from the filing of an application in the office of the Department of Water Resources. Concerning the use and appropriation of surface water in Nebraska, the State Constitution makes the following stipulations:

- The right to appropriate unappropriated water of natural streams for beneficial use shall never be denied except when demanded by the public interest;
- Between those using the water for the same purpose, priority of appropriation gives the better right, however, when the waters of any natural stream are not sufficient for all appropriators, those using the water for domestic purposes have preference over any other purpose, and those using the water for agriculture have preference over those using the water for industrial and manufacturing uses; and
- When junior appropriators assert a temporary priority by water use preference over senior appropriators, compensation must be paid.

Anyone intending to appropriate the public waters of Nebraska must file an application with the Department of Water Resources for an appropriation permit. No work may be initiated on the construction, enlargement, or extension of any associated diversion works until a permit has been granted authorizing the appropriation. In addition to the name and address of the applicant, the application must include the appropriation source, amount of water desired, purpose of water use, location of the diversion works, time required to

complete construction, and the estimated time to apply water to the proposed beneficial use. The application must also include additional information and facts that the Department may find necessary.

Upon receipt of the application and supporting information the Department will record the application and review its content for compliance with filing procedures. If the application is found to be defective it will be returned within 30 days after its receipt and an additional 30 days will be allowed to correct and refile the application. If a corrected application is not returned within 30 days it will be cancelled.

If there is unappropriated water in the source of supply, if the proposed appropriation is not detrimental to the public welfare, and if denial of the application is not demanded by the public interest, the Department will approve the application. The Department may approve the application for a shorter period of time for perfecting the proposed appropriation, or for a smaller quantity of water, or for a smaller amount of land, than applied for. The Department may also impose any other reasonable conditions considered necessary to protect the public interest.

Within 6 months after approval of an application, the applicant must file with the Department a map or plat showing the diversion project. The exhibit must show the source being appropriated, all proposed dams, dikes, reservoirs, canals, powerhouses, and other structures for the purpose of storing, conveying, or using water. Failure to file the necessary exhibit will result in forfeiture of the appropriation.

Also within 6 months after approval of an application for irrigation, power, or other useful purpose, the applicant must commence construction of the diversion works and must diligently pursue the work to completion unless temporarily interrupted by some unavoidable and natural cause.

Within 1 year from the date of application approval, one-tenth of the total work must be completed. Failure to comply with these provisions will forfeit the appropriation. When requested by the Department the applicant must furnish records and satisfactory evidence that construction has begun and is progressing.

The Department may extend for reasonable lengths of time any of the requirements for completing and perfecting an application for the appropriation of water. An extension must be requested by filing a petition with the Department and describing the reasonable cause. The Department will file a notice of the petition filing in at least one newspaper of general circulation in the county or counties of the requested appropriation. On its own motion or if requested by an interested person, the Department must hold a hearing on the issue. If a hearing is held, a notice must be filed with the applicant, the person requesting the hearing, and anyone requesting notification of the hearing, and it must be published in at least one newspaper of general circulation in the county or counties of the appropriation. If no requests for a hearing are received the Department may grant the requested extension.

Groundwater Rights. Permits for the use of groundwater are required only in groundwater control areas for all water uses except domestic, and in noncontrol areas for industrial uses that extract more than 3,000 acre-ft/yr.

The rationale and procedures for establishing groundwater control areas were defined in the Nebraska Groundwater Management Act of 1975. In the Act, the State Legislature recognized that the management and protection of groundwater were essential to the economic prosperity of the state. In geographic areas where groundwater levels were declining, or where shortages or groundwater pollution could potentially occur, management practices were considered necessary to protect the supply. The designation of groundwater control areas provides the necessary mechanism for implementing management controls. Whenever it has information indicating that a control area should be designated, the board of directors of a natural resource district will notify the water resources director.

After a hearing and completion of any studies or investigations found to be necessary, the director will issue an appropriate order. If it is determined that a control area should be designated, the director will consult with state agencies to determine the final boundaries of the control area. If the final boundaries include lands within one or more districts other than the district initiating the original request, the affected districts will be notified prior to issuance of the final order.

Before constructing a well for nondomestic use in a control area, an application must be filed with the Natural Resources District. Representatives of the District will review the application and forward it to the director with their comments and recommendations within 30 days after the application is filed. Information on the application must include the size, type, and estimated depth of the proposed well; the estimated capacity in gallons per minute; the location of the proposed well; and the place and purpose of use of water extracted by the well.

If the director finds that the application is incomplete or defective, it will be returned for correction. If the correction is not made within 60 days the application is cancelled. Thirty days after receiving a complete and properly prepared application, the director will either issue a permit with or without conditions, or the requested permit will be denied. In general, applications in control areas are denied only if the director finds, with the advice of the District, that the location or operation of the proposed well would conflict with any regulations or controls adopted by the District, or the proposed use would not be a beneficial use of water for domestic, agricultural, manufacturing, commercial, or power generation purposes.

A total of three control areas have been declared in Nebraska. None of the ROI is located in control areas. In the past, public meetings have been held in Harrisburg and Gering to discuss the possibility of establishing a groundwater control area in the Pumpkin Creek Valley in the ROI.

In noncontrol areas anyone proposing to annually use 3,000 acre-ft or more of groundwater for industrial purposes must apply to the Department for a permit. Before filing the application, the applicant or his representative must attend a conference with Department representatives to establish the scope and extent of hydrologic investigations considered necessary in support of the application. Results of the specified hydrologic investigations must then be submitted to the Department along with the application, and all plans and specifications. The application must include the amount of water requested, the nature and location of use, location of the proposed well,

dates for the beginning and completion of construction, and the expected date of initial water use.

Within 30 days of the receipt of an application, the director will accept the application as a completed application or return it for corrections. After an application has been accepted, the director will set a time and place for a public hearing. The hearing will be held within or in reasonable proximity to the area in which the wells would be located. The hearing will be scheduled within 90 days after the application is accepted by the director. The notice of hearing must appear 2 consecutive weeks in a newspaper of general circulation in the area of the proposed wells. The last notice must appear no later than 10 days prior to the hearing.

At the hearing the applicant must present all hydrological data and other evidence supporting the application. The applicant may negotiate with any user of water to obtain an agreement whereby the user waives any cause of action against the applicant, in exchange for financial payment, substitute water, or other compensation. The director will consider the agreements in determining whether to grant or deny the permit. Within 90 days after the hearing the director will issue an order either granting or denying a permit. A permit will be granted only if the director finds that the proposed groundwater withdrawal is in the public interest. In determining the public interest, the director will consider possible adverse effects on existing surface or groundwater users, the availability of alternative sources of supply, the economic benefit of the applicant's proposed use, the social and economic benefit of existing users of water in the area, any waivers of liability from existing users, and other factors reasonably affecting the equity of granting the permit. The director may grant a permit for less water than requested by the applicant. To protect existing users of water, the director may also impose reasonable conditions on the manner and timing of the groundwater withdrawals and on the manner of any transfer of groundwater.

Following a hearing a permit may be revoked or suspended if the director determines that the permit holder failed to withdraw the groundwater within 3 years from the date specified in the permit, the permit holder has withdrawn more groundwater than the amount specified in the permit, or the permit holder has violated any of the conditions specified in the permit.

Although a permit is not mandatory in a noncontrol area for a municipal water supply, a public water supplier may apply to the director for a permit to develop groundwater from a remote groundwater source and transfer the water to the municipality. The permit will establish a priority of use date, and it will ensure the protection of all well spacing requirements. Information to be supplied on the application includes the amount of water requested; location of proposed well(s); length, diameter, and capacity of pipelines; and the dates for start and completion of construction. The application must be filed with the director along with maps showing the location of all proposed wells.

On receipt of an application the director will prepare a notice of the application filing and the time and place for a hearing to receive public testimony. The notice must be published in a legal newspaper in each county in which the wellfield is to be located for 3 consecutive weeks. In general, the notice will be published within 4 to 5 weeks after receipt of the

application and the hearing will be scheduled approximately 60 to 75 days after receipt of the application. The hearing must be at least 30 days after the last publication of the notice.

Not less than 10 days before the hearing, any person who claims that his interests are adversely affected may file with the director written objections to the granting of the permit. Following the hearing, if the director finds that the withdrawal of groundwater requested is reasonable, is not contrary to the conservation and beneficial use of groundwater, and is not otherwise detrimental to the public welfare, he will grant a permit for the amount requested or for a lesser amount.

Wells constructed for municipal, domestic, agricultural, and certain industrial purposes do not require a permit. However, in addition to conforming to minimum spacing requirements all wells must be registered with the Department within 20 days after completion of well construction. A Certificate of Well Driller Form must accompany the Registration Form.

Information contained on the Registration Form includes the location of the well, date of its completion, the intended use the size and pumping capacity of the well, the number of any permit required for water use and any other information requested by the director. Information to be shown on the Certificate of Well Driller Form includes location, date of construction, method of sealing, type and lengths of casing and screen, pump test information, and an accurate log of the well.

Nebraska statutes provide for a minimum spacing of 600 feet between irrigation wells and 1,000 feet between irrigation or industrial and municipal wells. Anyone may apply to the Director of Water Resources for a special permit to construct a well without regard to the spacing requirements. The application must be on a form approved by the director and it must include location of the well, the reason for seeking a special permit, the proposed size of the well, its estimated capacity, and the name of the person selected to drill the well.

When considering the approval or rejection of a special permit application, the director will consider the known groundwater supply and the probable effect of the well on this supply and the surrounding land. The application may be approved or rejected in whole or in part, and the special permit issued or refused accordingly.

Temporary Water Rights. Water rights in Nebraska are granted for an indefinite period of time. Although the temporary use of an existing water right may be acquired by direct negotiation with an existing holder of a valid water right, there are no statutory procedures for acquiring a temporary right. Because Nebraska law prohibits changes in the type of use, it would, therefore, be unlawful to temporarily purchase water permitted for one use and use the water for another purpose.

Existing Rights and Water Exchanges. Applications for the appropriation of water must specify a purpose of use, a location for the point of diversion, and a location for the place of use. Therefore, the water right attaches to the land and when title to land is transferred to a new owner all water rights associated with that land are transferred also. In August 1983 a new Nebraska statute became effective that allows changes in the place of use. This statute is described in Section 2.6.2.2.1.3.

2.6.2.2.1.3 Constraints on Water Use

Beneficial Use. Beneficial use is the basis, measure, and limit of the right to use water in Nebraska. A Nebraska statute limits the maximum allowable use of water from the natural flow of streams for irrigation purposes to no more than the 1 cfs for each 70 acres of land and no more than 3 acre-ft each year for each acre of land. In addition, the amount used shall not exceed the least amount of water that experience may show is necessary for the production of crops, according to principles of good husbandry.

The Groundwater Management Act of 1975 was implemented by the Legislature because the beneficial use of groundwater was considered to be essential to the economic prosperity of the state.

In geographic areas where groundwater may be declining or where shortages may occur, the implementation of management practices to conserve and protect the supplies was considered essential.

Change in Type or Place of Use. Current Nebraska law does not allow for changes in the type of water use of an existing water right. In August 1983, a statute became effective that allows a water right to be severed from the land and transferred to a new location of use within the same river basin. Applications to transfer the location of water use must be filed with the Director of Water Resources. If the applicant is not listed with the Department as the appropriator a notice of change in ownership must also be filed. Before filing the application, written consent from all owners of lands involved in a surface water transfer must be acquired. If an application filed with the Department is incorrect or incomplete it will be returned for correction. A corrected application must then be filed within 30 days or the application will be cancelled.

Department representatives then review the application and records in the Department pertaining to the application, and identify additional information required in support of the requested change. The applicant is then notified of the additional information required. The review process, followed by requests for additional information, continues until all necessary information has been assembled. At a minimum, necessary information will include a list of all appropriators and/or lands that may be affected by the transfer, maps showing current place of use and proposed place of use, studies demonstrating that the proposed consumptive use of water will not exceed historic consumptive use, and the studies identifying both historic and future conveyance losses and locations where historic and future excess flows return to the water supply source.

When satisfied that all requested information has been submitted, the director will either schedule a public hearing and publish a notice of the hearing, or publish a notice describing the application and proposed transfer. Notices must be published once a week for 3 consecutive weeks in a newspaper of general circulation in each applicable county. Anyone wishing to object to the application filed, or request a hearing, must do so within 2 weeks from the last date of publication. Whenever requested, a hearing must be held. Following a hearing and if approval is granted, a map or plan must be filed with the director within 6 months. Failure to file a map or plan within the 6-month period will cause cancellation of the application for transfer of

use. The approval of the map or plan constitutes the final approval of the transfer, and water may not be applied at the new location prior to this approval.

Priority Based on Type of Beneficial Use. Beneficial use is the basis for all water rights in Nebraska. Water uses recognized as beneficial use include domestic, irrigation, industrial or manufacturing, and power. Between those using the water for the same purpose, priority of appropriation gives the better right. However, when the waters of any natural stream are not sufficient for the use of all those desiring to use the water, those using the water for domestic purposes have priority over those claiming it for any other purposes and those using the water for agricultural purposes have priority over all other beneficial uses. When junior appropriators assert a temporary priority by water use preference over senior appropriations, compensation must be paid.

The purchase of water from permitted wells in groundwater control areas would require approval of the natural resources district administering the control area. The purchase or temporary purchase of groundwater from permitted or nonpermitted wells in noncontrol areas is not regulated by the NDWR. The NDWR is aware that water from groundwater sources and natural lakes is temporarily sold on an informal basis. The NDWR has no objections to the temporary purchase and change in type or place of use of water from these sources. The only activity that would require NDWR approval would be use of the water outside of the state (Nebraska Department of Water Resources 1983b). Rule changes recently approved by the State Legislature include methods for the granting of permits authorizing the transfer of groundwater to an adjoining state. This procedure is discussed in the following section on interstate requirements.

Interstate Requirements. Rights of the use of water in interstate rivers and streams are generally regulated by interstate compacts ratified by Congress or by a decree from the United States Supreme Court apportioning the available waters among the affected states.

The waters of the South Platte River downstream from the west boundary of Washington County, Colorado, are apportioned by the South Platte River Compact between Colorado and Nebraska which became effective on March 8, 1926. The waters of the North Platte River Basin are allocated among the states of Colorado, Wyoming, and Nebraska by a decree of the Supreme Court of the United States dated October 8, 1945. It was modified by the Court in 1953 pursuant to a stipulation of the parties.

The compact and decree are administered by the officials of the states who are responsible for the administration of the water laws of their respective states.

Change in the place of beneficial use if the new location is not in the state of Nebraska has been restricted by Nebraska law. That law was challenged in the Supreme Court Ruling Sporhase vs. Nebraska on July 2, 1982, which ruled that states cannot outlaw the shipment of groundwater across their borders. A permit for use is required now under the same terms as any other permit for groundwater use in the state and on which the director will rule without considering reciprocal rights from other states but considering all other Nebraska controls on water use and type and place of beneficial use.

An additional Nebraska statute became effective in August 1983 that authorizes the transfer of groundwater to an adjoining state. Authorization must be requested by filing a petition with the Department. The petition must include a discussion of the location and capacity of all existing or reported wells, the proposed water use, the location of water use, an estimate of the annual water use in the adjoining state and the expected maximum daily withdrawal. A notice will then be published once a week for 2 consecutive weeks in a newspaper of general circulation in the Nebraska county(s) where the well(s) is located. Anyone objecting to the proposed transfer must file a written pleading with the Department within 10 days of the last publication of the notice. If there are no objections filed, the director will use his/her discretion on whether to hold a hearing. After the conclusion of the hearing, or based on information in the petition, the director will prepare a written order of findings and conclusions. A copy of the order will be delivered to each party of record.

Water Rights Versus Water Use. Lodgepole and Pumpkin creeks in the ROI are considered fully appropriated and no new surface water rights will be issued (NDWR 1983). The actual use of water is less than the surface water rights that exist.

It is expected that actual use of groundwater will be less than total permitted use since most permits are in terms of peak well delivery capability and wells are not used at those rates year-round.

2.6.2.2.2 Water Quality

2.6.2.2.2.1 Surface Water

The surface waters in Nebraska have been divided into five general beneficial use categories, with some categories being further subdivided.

The uses are:

1) Recreation

   a) Full Body Contact
   b) Partial Body Contact

2) Fish and Wildlife Protection

   a) Coldwater Habitat
   b) Warmwater Habitat
   c) Exceptional Warmwater Habitat
   d) Limited Warmwater Habitat

3) Water Supply

4) Agricultural

5) Industrial

Each stream segment and its tributaries have been assigned beneficial uses and each has been assigned specific criteria dealing with dissolved oxygen,

un-ionized ammonia, conductivity, fecal coliform, and chlorine residual. The beneficial uses and criteria for the streams in the ROI are given in Table 2.6-19.

In addition to the specific criteria given to the stream segments, a set of general criteria also apply to all segments in the state. These criteria cover aesthetic conditions, solids, toxics and deleterious substances, residue, oil, and floating substances. Table 2.6-20 gives the numerical standards for the general criteria.

State regulations contain an antidegradation clause. This applies to those waters which possess an existing quality which is better than necessary to maintain assigned beneficial uses. Under most circumstances, the existing water quality will be maintained and protected. However, the State may choose to allow lower water quality as a result of necessary justifiable and widespread economic or social development, and as long as such development does not degrade the water quality to the point it interferes with or becomes injurious to existing instream water uses.

2.6.2.2.2.2 Groundwater

Groundwater in Nebraska is protected by both an antidegradation clause and a set of water quality standards. The water quality standards are listed in Table 2.6-21. In situations where the existing water quality is better than the standards, the antidegradation clause will take preference to maintain the quality of the water.

The State of Nebraska has also published rules and regulations for underground injection control. These regulations govern the issuance of permits, underground injection practices, methods of injection, etc.

2.6.3 Surface Water Hydrology and Quality

Surface water within the ROI consists of all surface drainages from basin headwaters to drainageway main stems that could affect, or be affected by, the project. Principal drainageways in the ROI are the Crow, Lodgepole, Horse, Pumpkin, and Chugwater Creek ACSs. Crow and Lodgepole creeks are tributary to the South Platte River. Chugwater Creek is tributary to the Laramie River which like Pumpkin and Horse creeks are tributary to the North Platte River. The Platte River drainage is part of the Missouri River Basin.

Table 2.6-22 summarizes approximate area and land use in each watershed in the ROI.

This section provides information on existing streamflow, lakes and reservoirs, waste disposal practices, and water quality conditions for each of the ACSs.

The ROI is drained by predominantly eastward-flowing streams. Streamflow in the ROI is extremely variable with annual peak flows occurring in the spring or early summer and annual low flows occurring in late summer and winter months. Peak runoff is primarily derived from a combination of snowmelt runoff and the occurrence of heavy seasonal precipitation during April, May, and June. About half of the annual precipitation occurs during those months,

Table 2.6-19

CLASSIFICATION AND CRITERIA OF STREAMS OF INTEREST IN NEBRASKA

	Water Uses							Criteria				
	Recreation		Fish & Wildlife									
	Full Body Contact	Partial Body Contact	Cold Water	Warm Water Exceptional Warm Water	Limited Warm Water	Agricultural	Industrial	Water Supply	Dissolved Oxygen (mg/l) Not Less Than	Un-ionized Ammonia as N (mg/l) Not to Exceed	Conductivity (Micromhos) Not to Exceed	Fecal Coliforms Per 100 ml Geometric Mean Not to Exceed	Chlorine, Residual (mg/l) Not to Exceed
Pumpkin Creek Upper Reaches to Confluence with Lawrence Fork			X			X			5.6	0.04	1,500		0.01
Pumpkin Creek from Confluence of Lawrence Fork to Confluence with North Platte River			X			X			6.0	0.04	1,300		0.01
North Platte River from Confluence of Gering Drain to Confluence with Pumpkin Creek		X	X			X			6.0	0.04	1,300	1,000[a]	0.01
North Platte River from Confluence of Dry Spottedtail Creek to Confluence with Gering Drain		X	X			X			6.0	0.04	1,300	1,000[a]	0.01
North Platte River from Nebraska-Wyoming Border to Confluence with Dry Spotted-tail Creek at Mitchell		X	X			X			6.0	0.04	1,300	1,000[a]	0.01
Lodgepole Creek from Confluence of Crow Creek to Nebraska-Colorado Border					Limited	X			5.0	0.1	2,250		
Lodgepole Creek from Confluence of Sidney Draw to Confluence with Crow Creek					Limited	X			5.0	0.1	1,000		
Lodgepole Creek from Owasco Ditch (Diversion) to Confluence with Sidney Draw					X	X			5.6	0.06	625		
Lodgepole Creek from Nebraska-Wyoming Border to Owasco Ditch (Diversion)			X			X			5.6	0.04	900		

Note: a Nor shall fecal coliforms equal or exceed 2,000 per 100 ml in more than 10 of the samples.

Table 2.6-20

GENERAL NUMERIC CRITERIA
FOR SURFACE WATERS IN NEBRASKA

These criteria apply to all surface waters of the state with respect to man-induced conditions.

Parameter	Criteria
Hydrogen Ion	Hydrogen ion concentrations expressed as pH shall be maintained between 6.5 and 8.5 with a maximum total change of 0.5 pH unit from the value in the receiving waters.
Temperature	Temperature of receiving water shall not be increased by a total of more than 5°F (3°C) from natural outside the mixing zone. For the Missouri River, from the South Dakota-Nebraska state line near Fort Randall Dam to Sioux City, Iowa, maximum temperature limit is 85°F (29°C) with an allowable change of 4°F (2°C) from natural. For cold waters, the maximum limit is 68°F (20°C) with an allowable change of 5°F (3°C) from natural. For warm waters the maximum limit is 90°F (32°C). For impoundments, the temperature of the epilimnion of surface waters shall not be raised more than 3°F (2°C) above that which existed before the addition of heat of artificial origin. Unless a special study shows that the discharge of heated effluent into the hypolimnion will be desirable, such practice is not recommended and water for cooling should not be pumped from the hypolimnion to be discharged to the same body of water.
Solids	Turbidity caused by human activity shall not impart more than a 10-percent increase in turbidity, as measured in Jackson Turbidity Units, to the receiving waters.
Oil	Petroleum oil shall not exceed 10 mg/l.
Chlorides	Chlorides shall not exceed 250 mg/l except as noted in specific numeric criteria.

Table 2.6-21

GROUNDWATER QUALITY STANDARDS IN NEBRASKA

Maximum Contaminant Levels: The following standards are the maximum contaminant levels in groundwater for the specified contaminants. The standards apply to all groundwater which has an existing TDS concentration less than 10,000 mg/l. When a maximum contaminant level is exceeded by the existing contaminant level, as determined by the Department, the latter shall be used in place of the former, except where the existing concentration is the result of an identified point source of pollution.

Primary (Health)

Contaminant	Maximum Contaminant Level (mg/l)
Arsenic (As)	0.05
Barium (Ba)	1.0
Cadmium (Cd)	0.01
Fluoride (F)	2.4
Lead (Pb)	0.05
Mercury (Hg)	0.002
Nitrate-nitrogen (NO_3-N)	10.0
Selenium (Se)	0.01
Silver (Ag)	0.05
Chlorinated hydrocarbons	
Endrin	0.0002
Lindane	0.004
Methoxychlor	0.1
Toxaphene	0.005
Chlorophenoxys	
2,4-D	0.1
2,4.5-TP Silvex	0.01

Table 2.6-21 Continued, page 2 of 2
GROUNDWATER QUALITY STANDARDS IN NEBRASKA

Contaminant	Level (pCi/l)
Radionuclides	
Combined radium-226 and radium-228	5
Gross alpha particle activity (including radium-226 but excluding radon and uranium)	15
Gross beta particle activity	50

Secondary (Aesthetic)

Contaminant	(mg/l except as indicated)	
Color	15	Color Units
Copper	1	
Foaming Agents (detergents)	0.5	
Hydrogen Sulfide	0.05	
Iron	1.0	
Manganese	0.2	
Odor	3	Threshold Odor Number
pH	6.8-8.5	Standard Units
Zinc	5	
Oil and Grease	1	

Table 2.6-22

AREA AND LAND USE IN THE WATERSHEDS
IN THE ROI

Watershed	Land Use (sq mi)			
	Area	Rangeland	Cropland	Other[1]
Crow Creek				
Upstream of Cheyenne	280	240	10	30
Between Cheyenne and Carpenter	180	160	15	5
Lodgepole Creek				
Upstream of Oliver Reservoir	1,160	290	855	15
Between Oliver Reservoir and Sidney Draw	610	145	465	-
Sidney Draw	780	270	510	-
Horse Creek				
Upstream of Hawk Springs Reservoir	660	560	75	25
Hawk Springs Reservoir to North Platte	1,040	300	720	20
Pumpkin Creek to North Platte	1,020	510	410	100
Chugwater Creek to Laramie River	680	550	100	30
Drainages to North Platte From South				
Laramie River to Horse Creek	560	140	420	-
Horse Creek to Pumpkin Creek	160	40	120	-

Note: 1 Forest, mountains, urban.

consisting mainly of sporadic local thunderstorms. Numerous ephemeral streams flow exclusively during this period. Low flows during late summer, fall, and winter months derive primarily from groundwater discharge.

Storm events can cause a rapid response in streamflows leading to conditions of flooding. The potential for flood conditions is greatest during late spring and early summer when storm activity can combine with rapid snowmelt and storage reservoirs at or near capacity to create high streamflows. Localized flooding can also occur independently of these conditions due to intense rainfall and inadequate channel capacities.

Major lakes and reservoirs in and near the ROI can be seen in Figure 2.6-7. The principal purpose of reservoirs is storage for irrigation, although some are used for municipal water supplies and flood control. In addition to the reduction in peak flows which such facilities provide through storage and subsequent release, a net decrease in streamflows also occurs due to diversion of water for consumptive uses. Some water returns to streams via irrigation return flows and wastewater discharges.

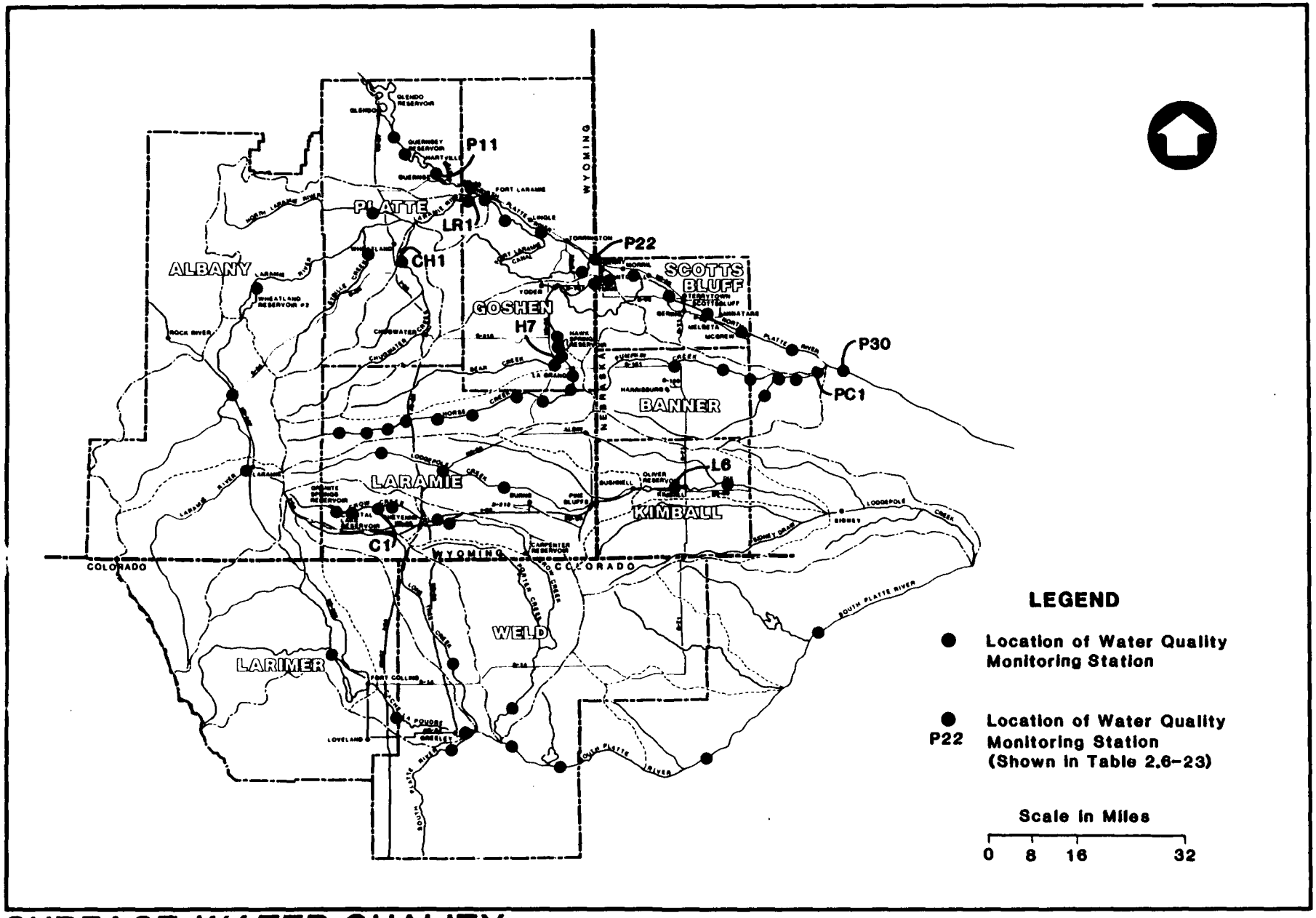
Another influence on surface flows is transbasin diversion of water, such as the Cheyenne Stage I Diversion Project. In such instances, surface flow is reduced in the basin from which water is diverted, and generally increased in the receiving basin. These effects do not necessarily balance due to losses and consumptive use.

A variety of water quality monitoring stations have been established in the ROI as shown on Figure 2.6-7. Selected stations are summarized in Table 2.6-23 to provide an overview of surface water hydrology and quality in the ROI.

The chemical quality of surface waters in the ROI is strongly dependent on that of the groundwater because of the interrelationships between the two. In most of the drainages the streams both gain water from the groundwater and lose water to it depending on the local hydrologic and geologic conditions in the stream reach. In the North Platte River, for example, the surface flow is predominantly gaining from the groundwater reservoir in most reaches (Gordon 1966).

The biological quality of the surface water is less dependent on groundwater than is chemical quality. Influences on the biological quality of surface water occur primarily at the land surface. These include such influences as those generated by man, animals (both wild and domestic), and the diverse populations of microorganisms that live near or at the land surface.

In general, the more frequently water is used and reused, the lower its quality. For example, when water begins flowing in the Laramie Mountains, either from runoff or as an effluent from a groundwater source, it may be exposed to soluble geologic material that increases the TDS, to erodible soils that may increase suspended material, and to humic material that may increase the color and organic content of the water. Agricultural water use can cause increased dissolved solids concentrations due to the evapotranspiration of water that concentrates dissolved materials. Also, agricultural water can be contaminated by chemicals such as fertilizers, pesticides, and herbicides. After return to the surface water or groundwater system the water flows down



SURFACE WATER QUALITY MONITORING STATIONS

FIGURE NO. 2.6-7

Table 2.6-23

SUMMARY OF WATER QUALITY
IN THE REGION OF INFLUENCE

		Crow Creek C1			Lodgepole Creek L6			Horse Creek H7			Laramie River LR1			Chugwater Creek CH-1		
	Units	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum
Alkalinity (as $CaCO_3$)	mg/l	101	191	242	216	232	245	-	-	-	95	208	287	-	-	-
Bicarbonate (HCO_3)	mg/l	123	232	295	257	282	299	272	299	376	110	253	356	209	306	339
Carbonate (CO_3)	mg/l	-	-	-	0	1	3	0.0	0.0	0.0	0.0	0.6	24	-	0	-
Calcium (Ca)	mg/l	57	66	76	53	63	70	52	61	77	28	77	88	62	93	107
Chloride (Cl)	mg/l	62	79	110	21	28	37	5.7	8.4	12	1.6	18	33	17	19	21
Discharge (Q)	cfs	11	16.8	27.5	0.71	4.6	10.	1.5	29.3	122	13.6	257	4,200	5.3	26.8	41.9
Fluoride (F)	mg/l	-	-	-	0.8	0.9	1.0	0.6	0.9	1.3	0.1	0.64	1.4	0.6	0.65	0.8
Magnesium (Mg)	mg/l	11	12	13	13	15	16	8.3	15	17	7.3	24	54	12	27	42
pH	Units	7.1	7.4	7.6	7.2	7.8	8.3	7.7	8.1	8.4	7.2	8.0	8.5	7.4	7.7	8.5
Potassium (K)	mg/l	0.3	7.4	11	7.3	9.2	12	5.6	6.9	8.8	1.0	5.2	11	6.3	7.0	12.0
SAR (Sodium Adsorption Ratio)		1.8	2.2	2.6	0.9	1.1	1.3	0.8	1.2	1.7	0.3	1.3	3.0	0.7	1.38	1.70
Silica (SiO_2)	mg/l	18	21	25	41	45	48	35	43	49	0	25.8	39	27	35	40
Sodium (Na)	mg/l	60	72	82	34	38	47	-	-	-	8	53	92	22	59	70
Specific Conductance	μ mhos/cm	690	884	1,515	530	592	674	504	564	635	365	756	960	546	865	913
Sulfate (SO_4)	mg/l	70	86	110	30	39	44	20	36	60	20	170	243	72	188	204
Turbidity	NTU	7	23	50	10	28	65	-	-	-	2	3.8	8	-	-	-
Temperature	°C	5.2	13.7	18.8	0	11.2	24	0.5	12.1	24.5	0.0	10.6	25.5	-	21.6	-
BOD	mg/l	-	-	-	1.7	6.3	10	-	-	-	-	-	-	-	-	-
COD	mg/l	60	127	190	-	-	-	-	-	-	-	-	-	-	-	-
Hardness (as $CaCO_3$)	mg/l	190	212	240	210	224	240	180	211	250	62	292	550	204	343	376
Dissolved Oxygen	mg/l	1.5	2.8	6.3	4.9	10.4	13.2	-	-	-	6.8	10.4	15.8	-	-	-
TDS	mg/l	391	474	542	339	384	423	323	369	415	242	502	640	366	602	642
Nitrate	mg/l	0.03	1.11	4.0	1.4	2.5	4.9	0.2	0.26	0.61	0.0	0.5	1.7	-	1.4	-
Ammonia	mg/l	5.4	12.8	18.5	0.07	1.33	3.8	-	-	-	-	-	-	-	-	-
Phosphorus	mg/l	1.8	5.8	72	0.24	1.28	3.0	-	-	-	0.0	0.04	0.4	-	-	-
Boron	μg/l	-	-	-	90	158	270	40	92	150	0	85	240	90	125	210
Iron	μg/l	120	452	980	20	130	370	0	11	120	0	183	1,500	-	50	-
Arsenic	μg/l	5.0	7.6	10	7.0	32	82	-	-	-	2	2.3	3	-	-	-
Copper	μg/l	22	38	60	4.0	11	20	-	-	-	1.0	7.0	17	-	-	-
Fecal Coliform	#/100ml	1.76×10^6	5.2×10^6	8.1×10^6	400	2.39×10^4	1×10^5	-	-	-	2	262	268	-	-	-

Table 2.6-23 Continued, Page 2 of 2
SUMMARY OF WATER QUALITY IN THE REGION OF INFLUENCE

		North Platte River P11			North Platte River P22			North Platte River P30			Pumpkin Creek PC1		
	Units	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum	Minimum	Average	Maximum
Alkalinity (as $CaCO_3$)	mg/l	88	136	173	130	200	276	15	231	277	17	213	256
Bicarbonate (HCO_3)	mg/l	107	166	211	157	244	336	220	285	338	275	288	297
Carbonate (CO_3)	mg/l	0.0	0.0	15	0.0	0.5	10	0.0	0.04	24	0	0	0
Calcium (Ca)	mg/l	26	62	103	34	72	99	53	78	95	16	58	191
Chloride (Cl)	mg/l	0.5	15	39	3.1	18	37	12	21	29	6	10.5	69
Discharge (Q)	cfs	2.4	308	7,060	164	1,071	8,430	263	1,155	5,699	1.0	18.6	65
Fluoride (F)	mg/l	0	0.48	1.2	0.0	0.47	1.3	0.2	0.5	0.8	0.6	0.68	0.8
Magnesium (Mg)	mg/l	8	23	48	2.7	21	43	14	20	24	4	16.6	65
pH	Units	6.9	7.7	8.6	7.2	8.1	8.5	7.1	7.9	9.0	7.9	8.4	10.9
Potassium (K)	mg/l	0.3	3.0	6.8	3.0	6.1	10.0	6.0	10	20	1.5	9.9	16
SAR (Sodium Adsorption Ratio)		0.7	1.5	3.5	1.1	2.0	3.6	1.5	2.3	2.7	1.1	1.6	2.6
Silica (SiO_2)	mg/l	1.5	11	17	0.0	20	44	0.3	3.5	44	39	44	53
Sodium (Na)	mg/l	20	55	150	37	77	121	53	86	110	13	47	73
Specific Conductance	μ mhos/cm	361	706	1,310	102	818	1,550	640	881	1,230	49	481	590
Sulfate (SO_4)	mg/l	83	204	479	110	202	270	134	197	250	9	38	99
Turbidity	NTU	-	-	-	1.7	19	270	4	53	230	3	29.4	125
Temperature	°C	0.0	13.1	27.5	0.0	11.7	25	0.0	14.8	305	0	13.8	33
BOD	mg/l	-	-	-	0.1	2.0	5.0	0.5	2.6	10	0.9	1.6	2.9
COD	mg/l	-	-	-	-	-	-	0.0	34	190	0	27	67
Hardness (as $CaCO_3$)	mg/l	150	248	450	171	267	520	206	277	330	110	190	240
Dissolved Oxygen	mg/l	-	-	-	7.1	9.6	12.6	5.6	9.7	15.2	7.4	10.3	13
TDS	mg/l	244	477	960	344	537	684	627	829	1,200	277	344	403
Nitrate	mg/l	0	0.46	3.2	0.02	1.1	5.6	1.8	2.5	3.1	0.9	2.2	4.2
Ammonia	mg/l	0.8	1.3	1.5	0.06	0.18	0.64	0.0	0.04	0.16	-	9.6	-
Phosphorus	mg/l	0.0	0.11	0.64	0.0	0.08	0.86	0.01	0.47	1.7	0.1	0.16	0.4
Boron	μg/l	0.0	90	440	0.0	108	590	90	139	190	50	104	130
Iron	μg/l	0.0	3.9	870	0.0	99	690	610	3,378	13,000	-	-	-
Arsenic	μg/l	-	-	-	-	-	-	5.0	7.7	11	-	-	-
Copper	μg/l	-	-	-	-	0	-	210	388	410	-	-	-
Fecal Coliform	#/100ml	-	-	-	0	1,256	21,000	0.5	159	1,600	0	152	200

Note: Stations are shown on Figure 2.6-7.

Source: STORET 1983.

gradient where it may be used for a municipal supply or be influenced by either municipal or industrial discharges. This same water may be used several times before it reaches the eastern edge of the ROI.

Seasonal variations in water quality are due to the natural climate changes and by man-induced variations, such as changes due to irrigation.

Erosion and sedimentation are also water quality problems. In general, water erosion is dependent on soil characteristics and topography, land cover conditions, and rainfall characteristics. The dominant soil characteristic is the susceptibility of the soil to erosion. The Universal Soil Loss Equation can be used to predict erosion, incorporating factors for rainfall, soil erodibility, slope, cropping, and conservation practices. However, for water quality planning, prediction of suspended sediment levels entering surface water is more important than prediction of on-site erosion.

Erosion is caused by wind and water action. The Geologic Resources EPTR discusses the relative magnitude of each for areas undergoing construction activity where a mitigation of 1 ton per acre of straw mulch cover is applied. With this assumed mitigation, typical total erosion for areas around cables and at F.E. Warren AFB is less than allowable soil losses of 4 to 5 tons per acre per year (T/acre/yr) for soils in the area. Totally unmitigated erosion at construction sites in the watersheds of interest would be much higher and a typical value of 75 T/acre/yr (EPA 1982) is used here to represent this condition for total erosion. Since the Universal Soil Loss Equation and methods of calculating wind erosion are highly specific to soil type and area, a more general analysis of existing erosion was performed to calculate existing erosion. Based on a recent report (Fly et al. 1983), average erosion rates of 3.1 tons per acre (T/acre) for cropland, 4.3 T/acre for rangeland, and 2.2 T/acre for forest and other land uses which occur in the area were used. Aerial photographs were used to estimate areas falling into each land use group. These areas were presented in Table 2.6-22.

Actual delivery of sediment to streams is a fraction of total erosion. Sediment delivery ratios are based upon drainage area and comparison to a reference area of 100 square miles (sq mi) with an assumed sediment delivery ratio of 0.2. For basins of greater size, the sediment delivery ratio is calculated by $(100/\text{area})^{0.2} \times 0.2$.

2.6.3.1 Crow Creek Watershed

The Crow Creek watershed comprises approximately 460 sq mi within the ACS with a length of 62 miles exclusive of meanders. Lone Tree and Lodgepole creeks drain the watersheds to the south and north of Crow Creek, respectively.

The Laramie Range of the Southern Rocky Mountains Physiographic Province forms the western side of the Crow Creek watershed. In many areas the topography has been eroded to a uniform, smooth, eastward sloping plain. In other areas the topography is irregular, with steep-sided valleys and rugged mountain peaks.

The area east of the mountains is within the High Plains section of the Great Plains Physiographic Province. The central portion of this area slopes eastward and has gently rolling topography of moderate relief, which is marked by

ephemeral and intermittent streams and underlain by Tertiary sediments. Surface gradients range from 100 feet per mile (ft/mi) to 20 ft/mi.

Major tributaries to Crow Creek within the ACS include South Fork Crow Creek, Middle Fork Crow Creek, North Fork Crow Creek, Diamond Creek, Clear Creek, and Dry Creek.

2.6.3.1.1 Streamflow

Four USGS recording gaging stations in the Crow Creek watershed within the ACS have long periods of record. These gaging stations are described as South Crow Creek near Hecla, Wyoming; Middle Crow Creek near Hecla; North Crow Creek near Hecla; and Crow Creek downstream of Cheyenne. The greatest floods of record for the gaging stations are: South Fork Crow Creek - 110 cfs on July 21, 1945; Middle Crow Creek - 496 cfs on September 8, 1933; North Fork Crow Creek - 3,600 cfs on June 2, 1929; and Crow Creek near Cheyenne - 8,200 cfs on June 5, 1929 (Lowham 1976).

Typical seasonal flow patterns for the recording gaging stations are shown in Table 2.6-24. Historic average annual streamflow for Middle Crow, South Crow, North Crow, and Crow Creek near Cheyenne are presented in Table 2.6-25.

Crow Creek is a gaining stream (receives water from the groundwater system) in the upper reaches and a losing stream (discharges water to the groundwater system) in the lower reaches. Seeps and springs in the western portion of Laramie County supplement surface flow in tributaries to Crow Creek. Pumping in the Cheyenne municipal wellfields has lowered the groundwater table to the extent that Crow Creek is presently dry during certain portions of the year between Silver Crown and Section 18, T14N, R67W.

Existing reservoirs capture a net average of 4,600 acre-ft/yr from the watershed runoff for delivery to Cheyenne (Section 2.6.1.1.6.1). As a result, flow below the reservoirs has been reduced. A pumping plant owned by the City of Cheyenne is located on the west side of F.E. Warren AFB where a flow of about 0.2 cfs is diverted from the creek and is used to fill Lake Pearson, Lake Terry, Lake Absaraka, Kiwanis Lake, and Sloans Lake in Cheyenne. This can divert the entire flow of Crow Creek upstream of Cheyenne during low flow periods.

A large spring yielding 2 cfs was developed in the Casper Formation in Section 8, T13N, R68W by the Union Pacific Railroad to supply water to steam locomotives. Presently most of the discharge from this spring flows into the South Fork of South Crow Creek and is diverted for irrigation. The aquifer from which this spring issues is probably recharged by seepage from Lone Tree Creek which flows across the Casper Formation where it outcrops (Lowry and Crist 1967).

In 1980, an average of about 8,000 acre-ft of wastewater per year was discharged into Crow Creek from the three Cheyenne area sewage treatment facilities, which are located in Sections 3 and 10, T13N, R66W. All discharges released from the Cheyenne area sewage treatment facilities flow to irrigation reservoirs. Releases from the reservoirs determine flow patterns downstream. All surface flow in Crow Creek ceases in Section 36, T12N, R63W, where it seeps into unconsolidated Quaternary terrace deposits. The Wyoming

Table 2.6-24

SEASONAL FLOW IN CROW CREEK

Average Monthly Flow (cfs)

Month	South Fork	Middle	North Fork	Cheyenne[1]
Jan	0.4	1.0	0.6	9
Feb	0.5	1.2	0.8	9
Mar	1.0	2.2	1.0	10
Apr	3.5	9.0	0.8	17
May	6.5	16.0	4.5	27
Jun	3.5	9.0	7.0	15
Jul	1.0	5.0	3.5	12
Aug	0.5	1.5	2.5	13
Sep	0.4	1.0	2.0	9
Oct	0.4	1.0	1.8	7
Nov	0.6	1.5	1.2	9
Dec	0.4	1.4	1.5	8

Note: 1 Includes impact of Cheyenne effluents, flow upstream of effluent typically 1 cfs which includes about 0.4 cfs of flow from an 84-inch storm sewer.

Source: USGS 1981.

Table 2.6-25

HISTORIC AVERAGE ANNUAL STREAM FLOW IN CROW CREEK WATERSHED

Gaging Station	USGS Station Number	Drainage Area (sq mi)	Period of Record	Average Annual Streamflow (acre-ft)
Middle Crow Creek Near Hecla, WY	06754500	25.8	1933-1969	3,700
South Crow Creek Near Hecla, WY	06755000	13.9	1933-1969	1,081
North Crow Creek Near Hecla, WY	06755500	27.0	1933-1948	2,442
Crow Creek	06756000	297.0	1923-1957	8,660

Source: USGS 1983.

State Engineer has made seepage measurements on Crow Creek during a period of high runoff and found that Crow Creek can lose as much as 13 cfs to the groundwater system before reaching Carpenter, Wyoming. A spring in the Carpenter area provides a small flow to Crow Creek.

Flooding is seen as a major problem in the Cheyenne Urban Area. The principal flooding problems are due to major thunderstorms which can occur from late spring through early fall. At the time of the studies, no major flood protection existed on Crow Creek. Although other reservoirs in the ROI are used as flood control reservoirs, reservoirs in the Crow Creek drainage area are only used for irrigation and water supply and have little or no effect on floods in the Cheyenne area.

Only the Dry Creek area has been the subject of a detailed flood analysis. Because of a lack of information on existing conditions related to flooding it has been necessary to develop procedures to calculate flood flows. A review of past flood investigations is followed by a description of methods used to evaluate flood potential.

FEMA flood insurance studies have been completed for both the incorporated and unincorporated areas of Laramie County. The studies are divided into five major parts: introduction, scope, engineering methods, floodplain management applications, and insurance applications. The major product of the studies is a floodplain map of each study area. The maps delineate the 100 and 500-year floodplains along Crow, Clear, and Dry creeks.

A statistical approach for determining flood insurance premiums and damages facilitated the use of standard recurrence interval flood events. The 10, 50, 100, and 500-year flood events were analyzed. The hydrologic analysis was geared toward determining the peak discharge and utilized a regional frequency approach developed by the USGS. A standard Log-Pearson statistical approach could not be employed because of a lack of gage records. Frequency-discharge, drainage area relationships were developed for Crow Creek. The hydraulic analyses were completed using field data to derive parameters for computer simulation. The program HEC-2 developed by the COE was used to determine flood peak elevations and hence areas of inundation.

In February 1979, BRW/Noblitt and Wright-McLaughlin Engineers completed a study on development of Dry Creek. The study analyzed flooding potential and existing problems in the Dry Creek Basin and included a hydrologic and cost-benefit analysis to evaluate different flood control alternatives. The goal of the Dry Creek Master Drainage Plan is to mitigate future flood damages and systematically reduce existing flood damage potential. Preventive actions include floodplain regulations, acquisition of flood-prone areas, floodplain information, and upstream (on-site) detention. Corrective actions include channel modifications, removal of flow constrictions such as undersized culverts, and a large scale flood control storage facility.

The Dry Creek Basin is 12.8 sq mi in size. It presently contains large segments of urban development in the upper and central portions. Development is projected in the foreseeable future to the point that the basin will be urbanized from the eastern boundary of F.E. Warren AFB to Highway 30. Projected development could have significant impacts on the hydrology of Dry Creek. If no means are undertaken to reduce runoff from the developing areas, the rate

of flow upstream of Highway 30 would be about 65-percent greater than it is presently for the storm which has a 1 percent chance of occurrence in any given year. This increased runoff would far exceed the capacity of existing culverts and greatly increase the amount of flood damage.

Flooding problems currently exist in several reaches of the major drainageways of Dry Creek. Evers Boulevard has a continual flooding problem since the street is located in the historic channel. The Buffalo Ridge development near the Dry Creek channel is in a hazardous location due to limited street and channel capacity in the vicinity. The culverts at Highway 30 restrict the flow and pose a threat of upstream flooding. Local problems exist throughout the basin where the lack of drainage swales and storm sewers result in flow down streets exceeding the carrying capacity.

Flood damages were calculated for the main reaches of Dry Creek. The damages were converted to an annual basis in order that a comparison of costs and benefits could be made. Over $700,000 of annual flood damages could occur for all of Dry Creek for the baseline future development, assuming no drainage controls. Possible financing of alternative preventive or corrective measures was discussed.

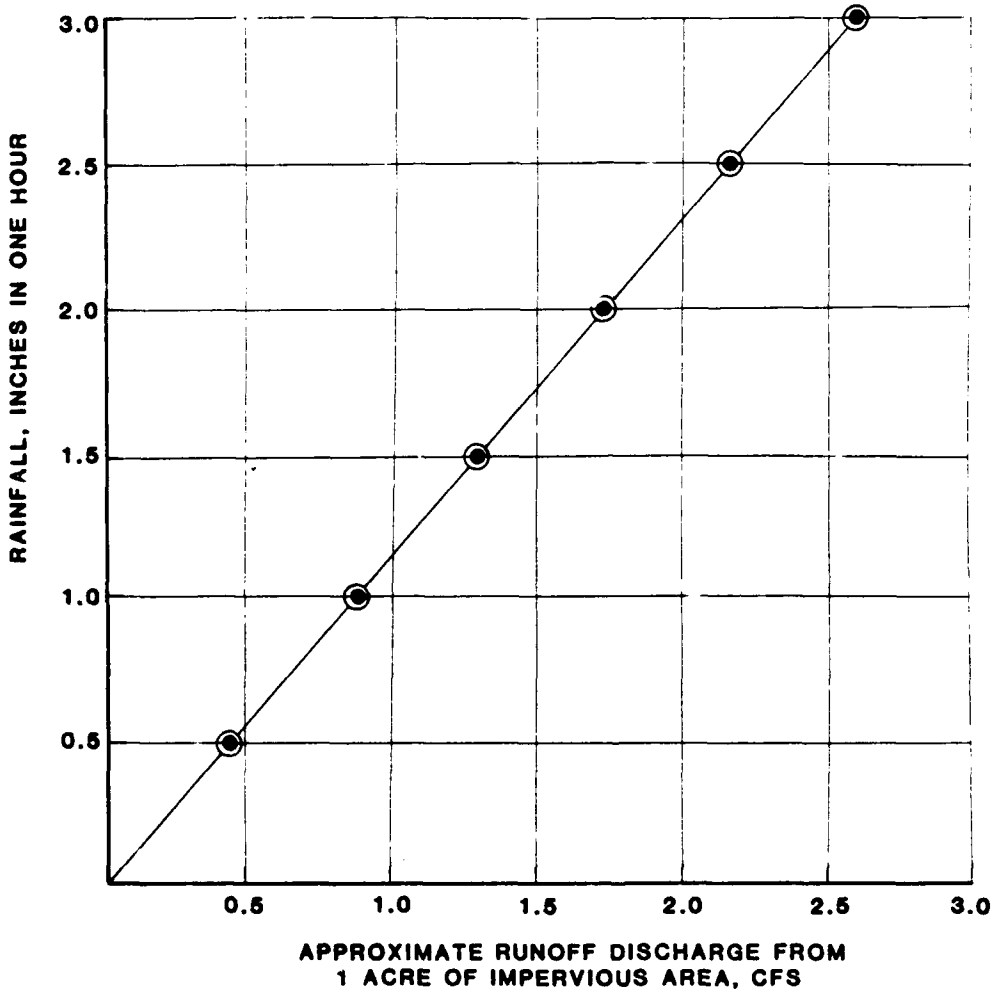
F.E. Warren AFB is in the Crow Creek drainage. Of the base's total 5,900 acres, more than half is unimproved. A portion of the improved area, probably about 1,000 acres, is served by several small storm drainage piping networks, which collect stormwater from streets. These storm drains discharge into Crow and Diamond creeks. In addition, considerable overland flow area lies tributary to these creeks, both on and off the base.

No official designation of flood-prone areas was found in the existing data. An estimate of the floodplain of Crow and Diamond creeks indicates the floodplain to be about 160 acres and shows it to be about 400 feet wide along 3 miles of Crow Creek and 1.5 miles of Diamond Creek. This estimate was not field-checked but appears reasonable for a preliminary value.

F.E. Warren AFB lies in Region 3 of the Western United States Precipitation Atlas. Precipitation charts for this region by the U.S. Bureau of Reclamation and NOAA show point values for precipitation (thunderstorm events) can be as much as 12 inches per hour. These high intensities are only used for drainage design when a large loss of life or property is possible. Maximum rainfall values of 3 or 4 inches per hour have been used for design of drainageways and storm drainage facilities.

Current information indicates that proposed new facilities including appurtenant roads and parking would add about 50 acres of impervious area at F.E. Warren AFB. An estimate of the impact of the additional impervious area can be obtained by considering the contribution of each additional acre of impervious area. Figure 2.6-8 shows that runoff values increase dramatically as rainfall intensity rises.

The existing flooding potential, or more correctly, peak flow potential for this study was evaluated by separate methods in the urban and nonurban areas. In the urban area with pipe systems, the modeling was completed using the Storm Water Management Model (SWMM) program. Inflows to Crow Creek were evaluated and hydrographs produced at the two storm sewered area outfalls. The TR-55 and TR-20 procedures were used for the remaining area.



SOURCE:

DRAFT SITE ANALYSIS,
BASE COMPREHENSIVE PLAN

RELATION BETWEEN RAINFALL AND RUNOFF

FIGURE NO. 2.6-8

The storm events modeled were the 2-year, 24-hour storm; 10-year, 3-hour storm; 50-year, 6-hour storm; 50-year, 12-hour storm; and the 100-year, 24-hour storm. These were selected because of the ranges of different storm runoffs which were produced from each of these storms. For each sub-basin, a hydrograph was developed by using 30-minute time increments to estimate peak flow for each period. A synthetic dimensionless hydrograph was applied to approximate runoff due to that interval's rainfall. These were then added together sequentially for all intervals to obtain the resultant storm hydrograph. The synthetic hydrograph is developed using SCS Technical Release 20 as described in Appendix A, Section A.3. The area, slope, flow length, soil type, and ground cover of each basin were determined using USGS topographic maps and land use data available for various neighborhoods in the Cheyenne Urban Area (Figure 2.6-9).

Each sub-basin hydrograph was routed to a designated point based on time of travel. The sub-basin hydrographs were combined to obtain the flood hydrograph for the basins by superposition of 30-minute time steps with the initial time step being the beginning of the storm for each sub-basin. A peak value and hydrograph was then produced for Crow Creek at its confluence with Diamond Creek.

Table 2.6-26 shows the peak values at the confluence of Crow Creek with Diamond Creek for each of the given storms.

Table 2.6-26

PREDICTED EXISTING FLOOD FLOWS IN CROW CREEK
UPSTREAM OF CHEYENNE

Storm	Total Rainfall (inches)	Hourly Intensity (average inches/hr)	Peak Flow (cfs)
2 yr - 24 hr	1.68	0.07	1,250
10 yr - 3 hr	1.71	0.57	1,815
50 yr - 6 hr	2.9	0.48	7,741
50 yr - 12 hr	3.3	0.28	9,710
100 yr - 24 hr	3.6	0.15	9,060

Since none of the reservoirs in the Crow Creek basin are operated with consideration to flood control, the assumption is made that the reservoirs are full at the beginning of the storm and that flood peaks are not attenuated by storage. Table 2.6-27 shows sub-basin flood peaks of the Cheyenne Urban Area for the 10-year, 3-hour storm; 2-year, 24-hour storm; and 50-year, 6-hour storm. Figures 2.6-9 and 2.6-10 show the corresponding sub-basins and designations.

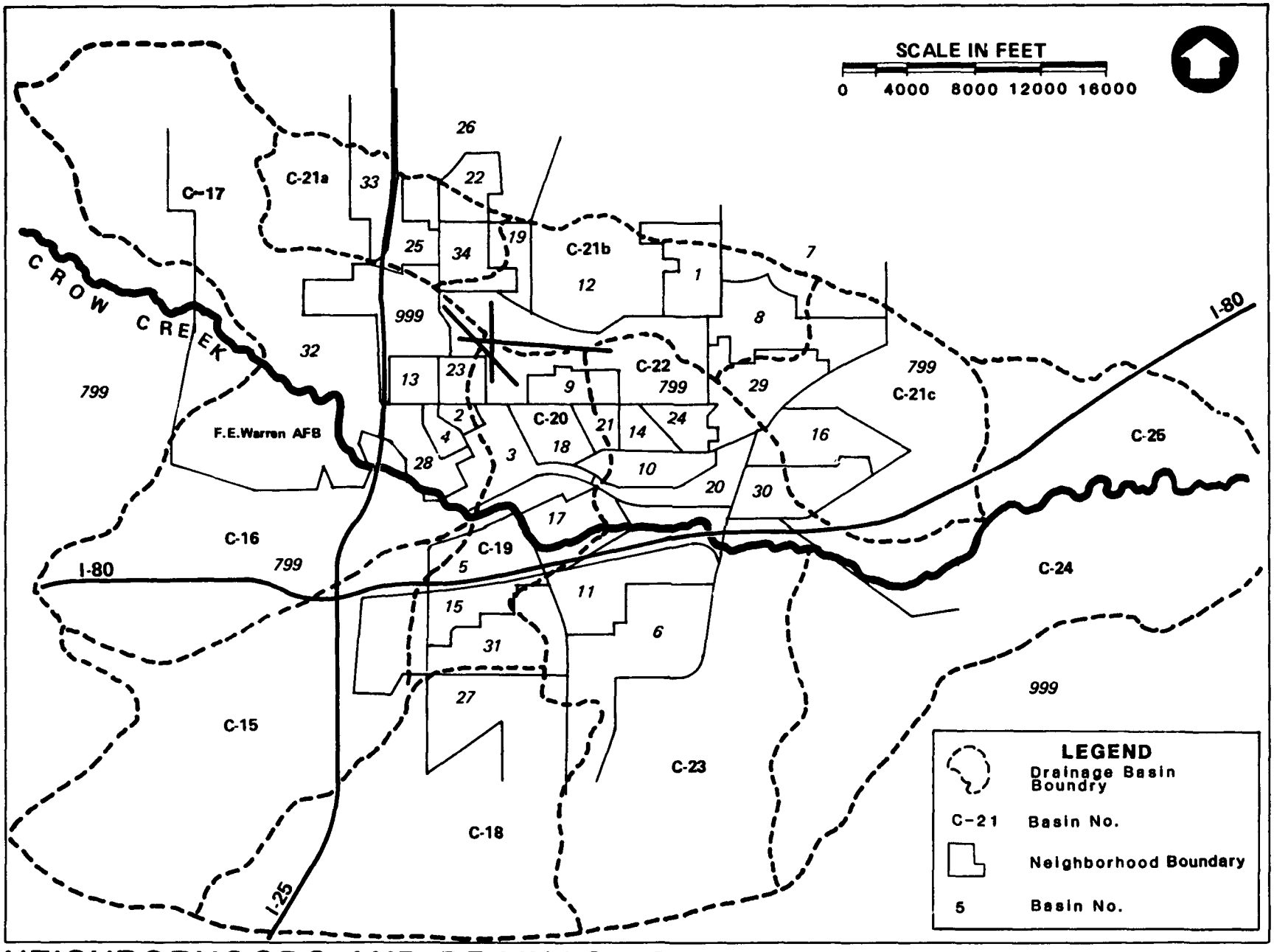
Sub-basins C17a and C20a were modeled using SWMM (Utilities EPTR) and had flows and peak times generated. The basins, because of their urban nature, have very high peak flows and very short peak times. As indicated in Table 2.6-27, the peak flows and recession limbs have substantially passed through the Crow Creek channel before the peak flow of the upper Crow Creek

Table 2.6-27

EXISTING FLOOD FLOWS IN
CHEYENNE URBAN AREA

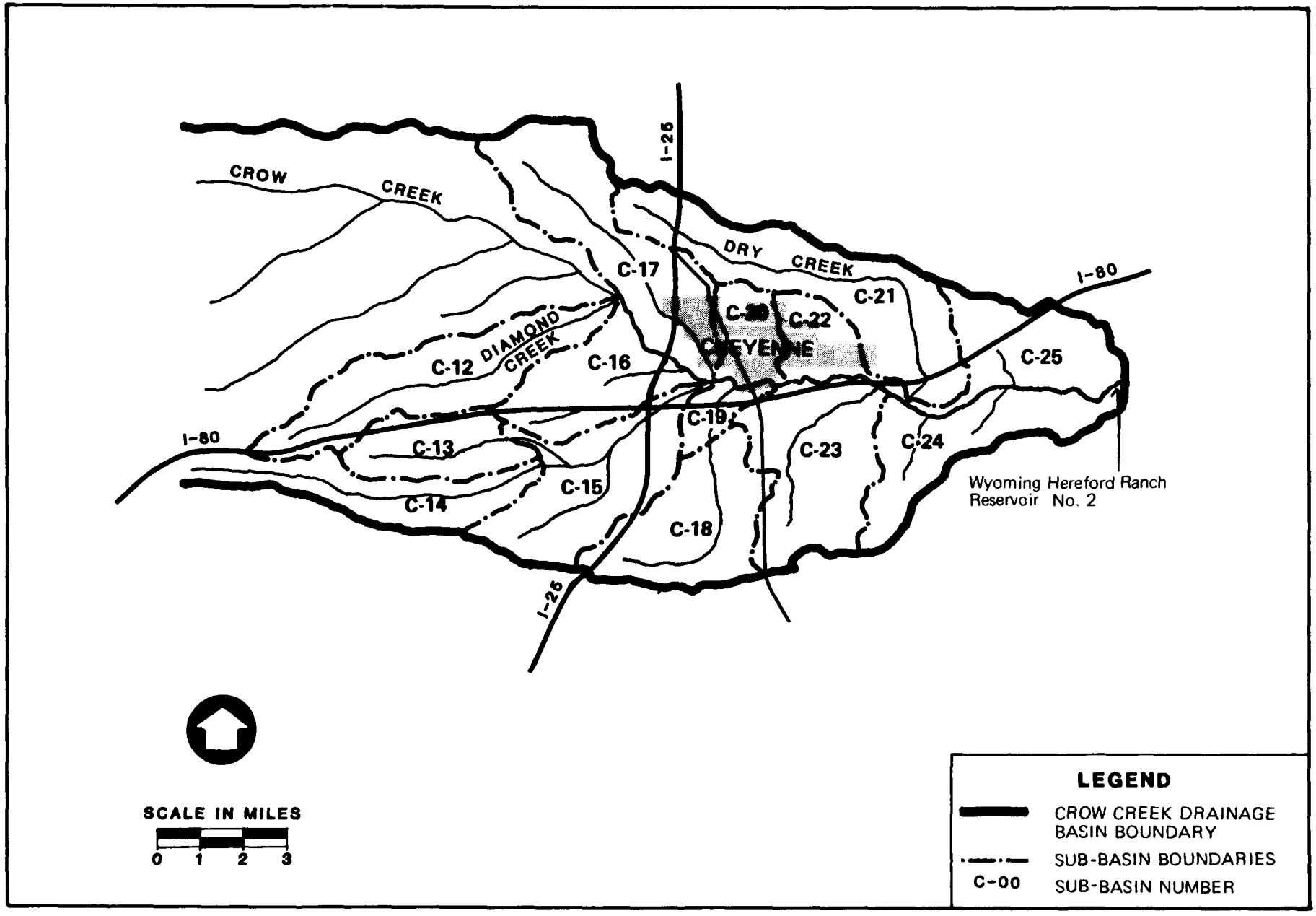
Basin	Name	Area (sq mi)	Curve Number	2 year, 24 hour Time to Peak (hours)	2 year, 24 hour Peak Flow (csf)	10 year, 3 hour Time to Peak (hours)	10 year, 3 hour Peak Flow (csf)	50 year, 6 hour Time to Peak (hours)	50 year, 6 hour Peak Flow (csf)
Crow Creek at Diamond Creek		248	-	34	1,279	18	1,815	20	7,741
C12	Diamond	10.89	70	30	37	19	42	21	194
C13	Clear Creek	4.25	70	29	27	8	42	10	189
C14	Clear Creek	9.51	70	29	33	19	38	20	173
C15	Clear Creek	10.28	71.5	22	62	12	79	13	332
C16	West Cheyenne	7.57	72.2	19	61	8	94	10	374
C17	West Cheyenne	10.83	72.4	25	64	14	78	16	312
C17a[1]	Cheyenne	1.20	-						
C18	South Cheyenne	8.73	71.1	18	66	8	108	9	451
C19	South Cheyenne	2.00	76.3	16	26	6	58	7	181
C20	East Cheyenne	0.74	81.6	16	15	6	34	7	87
C20a[1]	East Cheyenne	1.27	-						
C21a	Dry Creek	2.97	70	18	30	7	54	9	190
C21b	Dry Creek	5.07	74.6	19	52	8	84	10	296
C21c	Dry Creek	5.91	70	18	58	8	97	9	350
C22	East Cheyenne	4.76	78.9	22	59	11	81	13	246
C23	South Cheyenne	9.58	74	19	79	9	119	10	466
C24	Kingman Ditch	7.66	70	22	39	12	50	13	228
C25	Bolla Ditch	5.82	70.8	18	42	8	66	9	280
Crow Creek at Hereford Ranch Reservoir		346	-	40	1,503	23	2,096	26	9,068

Note: 1 No curve number developed for this basin - SWMM model utilized. Peak flow shown at Hereford Ranch Reservoir does not show effects of basins C17a and C20a due to short peak times. 120 and 300 cfs are the maximum capacity of the respective outfalls.



NEIGHBORHOODS AND DRAINAGE BASINS IN THE CHEYENNE URBAN AREA

FIGURE NO. 2.6-9



DRAINAGE IMPACTING FLOOD FLOW IN THE CHEYENNE URBAN AREA

FIGURE NO. 2.6-10

Basin has arrived. Consequently, the basins have little impact on regional flooding, while having some impact on local flooding (Utilities EPTR).

The peak flow of the 10-year, 3-hour storm is increased by about 280 cfs as it passes through the Cheyenne Urban Area. If this flow is compared to the capacity of the urban area structures (Table 2.6-28), one can easily see the structures which are undersized. The 2-year, 24-hour storm should present few if any regional flooding problems.

Another method for evaluating peak flow is the Lowham method (Lowham 1976). Lowham developed regression equations for estimation of mean annual flow and peak flow for different return periods. The method is used here as a check for the peak values calculated using TR-55 for the 50-year and 100-year events, P_{50} and P_{100}, respectively.

The Crow Creek Basin is primarily in Region 3 (75%), with a small amount in Region 4 of the hydrologic regions (25%). The following equations are applicable.

Region 3	Region 4	
$P_{50} = 1070\ A^{0.37}$	$P_{50} = 170\ A^{0.58}$	A = 248 sq mi
$P_{100} = 1480\ A^{0.37}$	$P_{100} = 242\ A^{0.57}$	

Values of 9,260 cfs for P_{50} and 12,780 cfs for P_{100} are calculated. The discrepancy of the Lowham values and the routed values can probably be attributed to two aspects. First, the Lowham method makes no consideration of routing time of basin peak. The method probably assumes a worst case in which all water within the basin peaks simultaneously. Second, no consideration of storm duration is made. In this light, the flows appear more reasonable. They are still approximately twice the flood flow values that were calculated for FEMA studies.

Although use of the Lowham method is predicated on lack of development of the basin, such as no dams, reservoirs or diversions, the method is still probably valid for Crow Creek because of the small amount of development, and because the existing reservoirs are not operated with consideration to flood control. Table 2.6-28 gives capacity information necessary for routing of the flood flows through the major structures on Crow Creek and its tributaries.

Also included are the following structures with associated dimensions.

Middle Crow Creek at Happy Jack Road, Two 7'x7' Box Culverts
Section 23, T14N, R69W

South Crow Creek at Happy Jack Road, Two 7'x7' Box Culverts
Section 23, T14N, R69W

Crow Creek at Round Top Road, 4-Span (4 @ 26') Bridge
Section 34, T14N, R67W

On comparison of the bridge capacities to the 100-year flood flows, it is immediately apparent that only a few, if any, of the structures would survive the 100-year event. Figure 2.6-11 shows the locations of structures 1 to

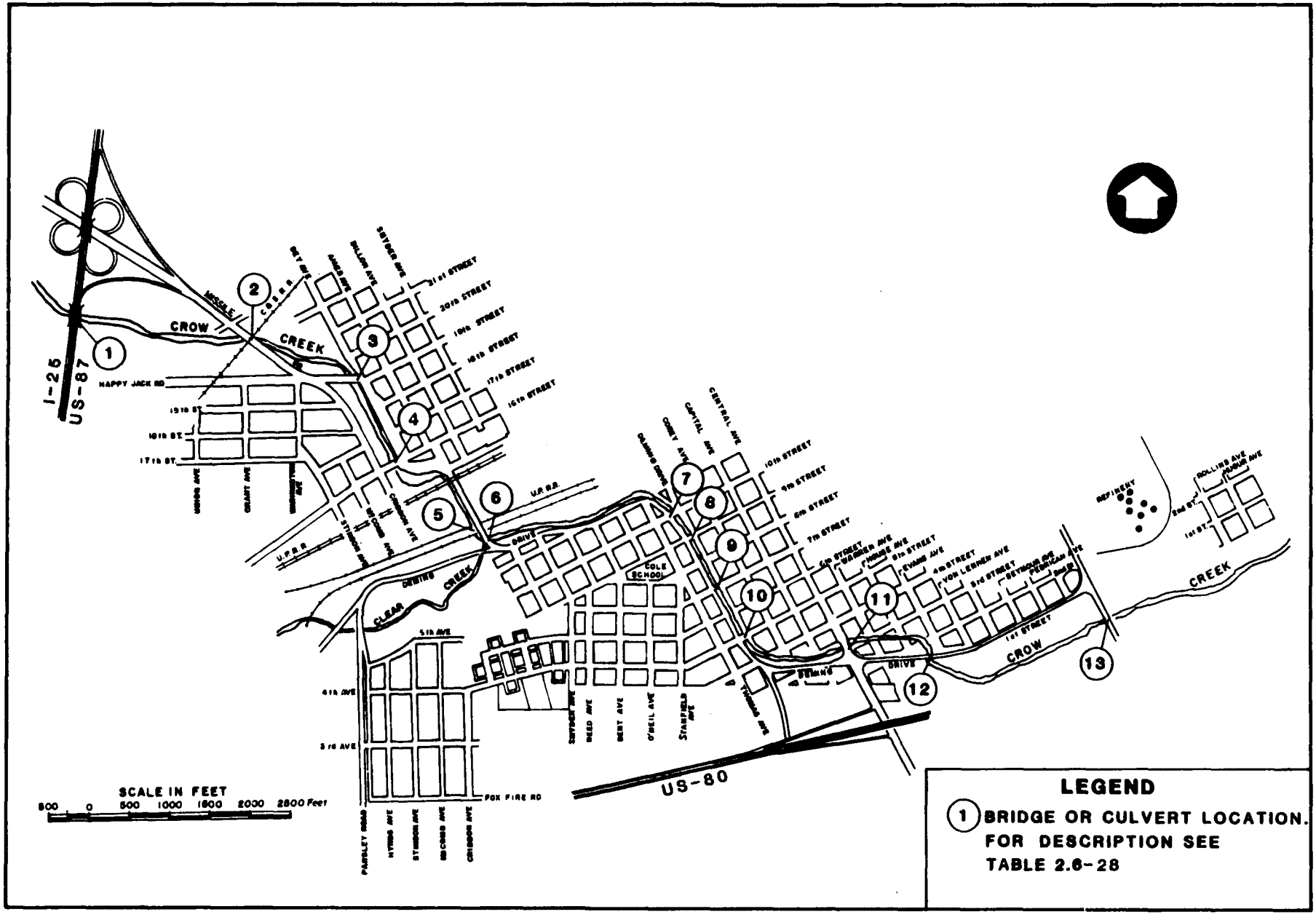
Table 2.6-28

EXISTING BRIDGE OR CULVERT CAPACITIES
CROW CREEK

	Location[1]	Size and Type	Capacity (cfs)
1.	I-25	5 span (1 @ 22.5', 3 @ 27' & 1 @ 22.5') Bridge	8,550
2.	Missile Drive	4-15'x15' Box Culverts	9,900
3.	19th Street[2]	5-14'x9' CMP Arch Culverts	3,250
4.	16th Street	1-Span (83') Bridge	6,700
5.	UPRR[3]	2-Span (2 @ 40') Bridge	4,500
6.	Deming Drive[4]	5-Span (1 @ 27', 3 @ 14' & 1 @ 22') Bridge	2,740
7.	Tenth Street[5]	4-Span (1 @ 25', 1 @ 30', 1 @ 19' & 1 @ 20') Bridge	4,580
8.	Ninth Street[6]	7-65"x40" CMP Arch Culverts	644
9.	Seventh Street (Ped)	2-Span (2 @ 60') Bridge	5,900
10.	Fifth Street	4-Span (4 @ 17') Bridge	6,320
11.	Central Avenue	5-10'x10' Box Culverts	5,500
12.	First Street	5-13'x9' CMP Arch culverts	3,700
13.	Morrie Avenue[7]	3-48" Concrete Pipe Culverts	450

Notes:
1 See Figure 2.6-11
2 The 19th Street culverts are planned to be replaced with a larger capacity structure around the year 1990.
3 The capacity of the UP Railroad bridge has been decreased by half because of street construction.
4 Deming Drive is a timber structure with load limits but is not planned to be replaced.
5 The Tenth Street bridge has been removed.
6 The Ninth Street culverts have been replaced with a 3-span bridge of 4,500 cfs capacity.
7 The Morrie Avenue culverts have been washed out. A bridge of 6,500 cfs capacity is proposed, but funding is not available.

Source: Banner 1983b.



CROW CREEK IN THE CHEYENNE URBAN AREA

FIGURE NO. 2.6-11

13. Structures 1 and 2 will probably survive the 100-year flood flow. All others would probably have some type of inundation. The Middle Crow, South Crow and Crow Creek structures will probably be adequate to pass a major flood. Comparison of the major recorded events with these capacities gives good indication that had those structures existed during the flood, they would have proven to be adequate.

2.6.3.1.2 Lakes and Reservoirs

Lakes, ponds, and reservoirs within the Crow Creek watershed range in size from small stock watering dugouts to large municipal and irrigation storage reservoirs. A list of the larger storage reservoirs in the Crow Creek watershed ACS is presented in Table 2.6-29. Hydrologic characteristics and water quality conditions for lakes, ponds, and reservoirs are generally not available. Small stock watering ponds and reservoirs, although numerous, are not believed at this time to have a significant effect upon surface water hydrology.

The Granite Springs Dam was completed in 1904, and is constructed of granite blocks laid in cement mortar. A main source of information on the Granite Springs Reservoir is the "Phase I Inspection Report, National Dam Safety Program, Granite Springs Dam" (COE 1978). The report contains area-capacity curves which provide information on capacity, stage-storage, and surface area-stage (Figure 2.6-12).

Outlet capacity is provided by a 24-inch diameter outlet pipe and an emergency spillway. The spillway capacity is about 760 cfs, or about equal to the 50 or 100-year flood event. Some spillway deterioration has been identified. An indication of the integrity of the dam as a whole is given in the COE summary. Basically stated, the COE found the dam to be structurally sound but hydraulically inadequate, and a major flood event could cause dam failure.

Granite Springs is operated as a supply reservoir to keep downstream Crystal Reservoir at capacity. Transfer from Granite Springs Reservoir is approximately equal to withdrawal from Crystal Lake Reservoir. All water transfers from the North Platte drainage (Stage I) are stored in Granite Springs and Crystal Lake reservoirs.

The Crystal Lake Dam was completed in 1910, and is a concrete masonry arch type dam. The "Phase I Inspection Report, National Dam Safety Program, Crystal Lake Dam" (COE 1978) contains area-capacity curves which provide information on capacity, stage-storage, and surface area-stage relationships (Figure 2.6-13). Since the capacity does not agree with records from the City of Cheyenne, a second set of area-capacity curves is also presented (Figure 2.6-14).

Outlet capacity is provided for by three 24-inch outlet pipes and an emergency spillway. The pipes have downstream control at a valve house at the base of the dam. The spillway is of broad crested weir type with a maximum capacity approximately equal to the 100-year flood.

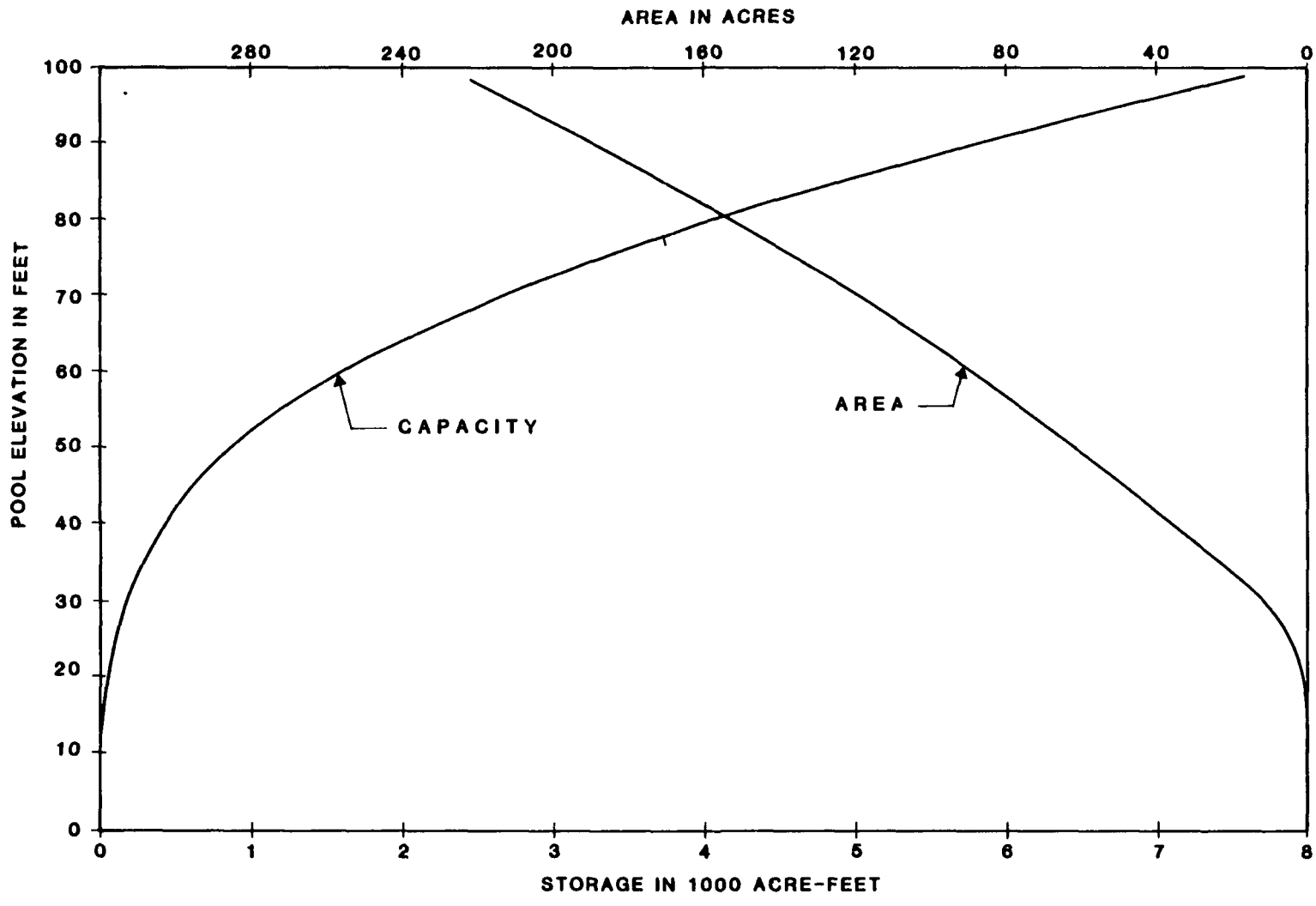
The integrity of the dam was investigated by the COE several years ago and again recently by Woodward-Clyde Consultants (Woodward-Clyde 1982). The report suggests that renovation of the dam and abutment will be necessary for the dam

Table 2.6-29

CROW CREEK WATERSHED
SUMMARY OF LAKES, PONDS, AND RESERVOIRS

USGS 7-1/2 Minute Quad Sheet	Reservoir, Lake, or Pond Name	Section	Township	Range	Storage Capacity (acre-ft)
Altvan	WY Hereford Ranch Res. #1	2	13N	66W	878
Archer, WY	WY Hereford Ranch Res. #2	3	13N	65W	877
Corie, WY	Name Unknown	17	13N	67W	-
Corie, WY	Name Unknown	18	13N	67W	-
Cheyenne South	Swan Reservoir	15	13N	67W	-
Cheyenne North	Lake Pearson	23	14N	67W	-
Cheyenne North	Lake Terry	23	14N	67W	-
Cheyenne North	Lake Absaraka	24	14N	67W	-
Cheyenne North	Kiwanis Lake	24	14N	67W	-
Cheyenne North	Sloans Lake	30	14N	66W	-
Cheyenne North	Lake Minnehaha	32	14N	66W	-
Hecla, WY	Granite Springs Reservoir	16	14N	70W	5,321
Hecla, WY	Crystal Lake Reservoir	26	14N	70W	3,410[a]
Hecla, WY	North Crow Diversion Res.	36	14N	70W	98
Hecla, WY	Schuyler Reservior	4	14N	69W	-
Hecla, WY	South Crow Creek Reservoir	1	13N	70W	12
Roundtop Lake, WY	Roundtop Lake	9	14N	67W	-
Silver Crown, WY	Gilchrist Reservoir	13	14N	69W	-
———	North Crow Reservoir	30	15N	70W	1,865

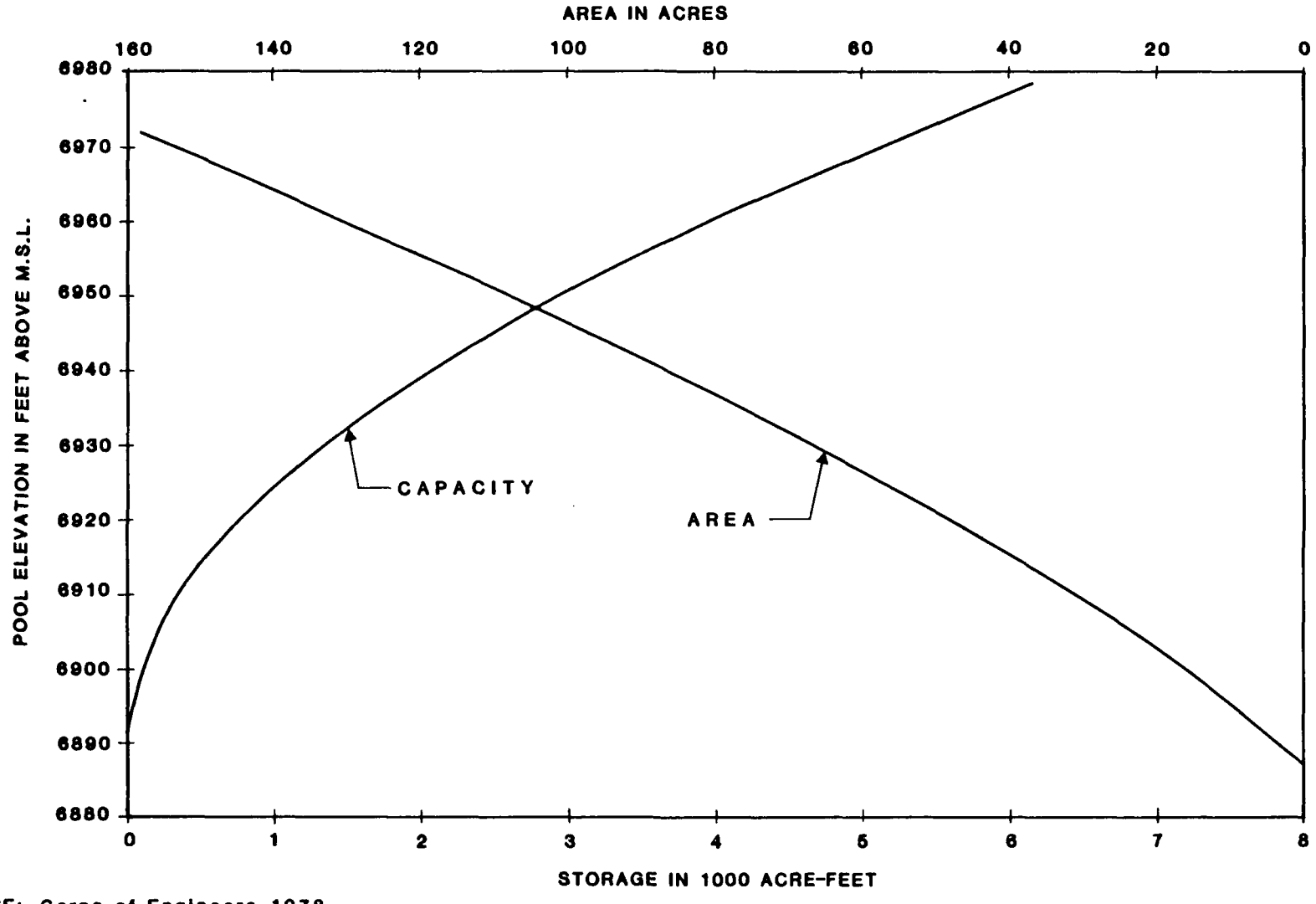
Note: a A current agreement with the State Engineer limits capacity to 2,962 acre-ft.



SOURCE: Corps of Engineers, 1978

GRANITE SPRINGS RESERVOIR AREA-CAPACITY CURVES

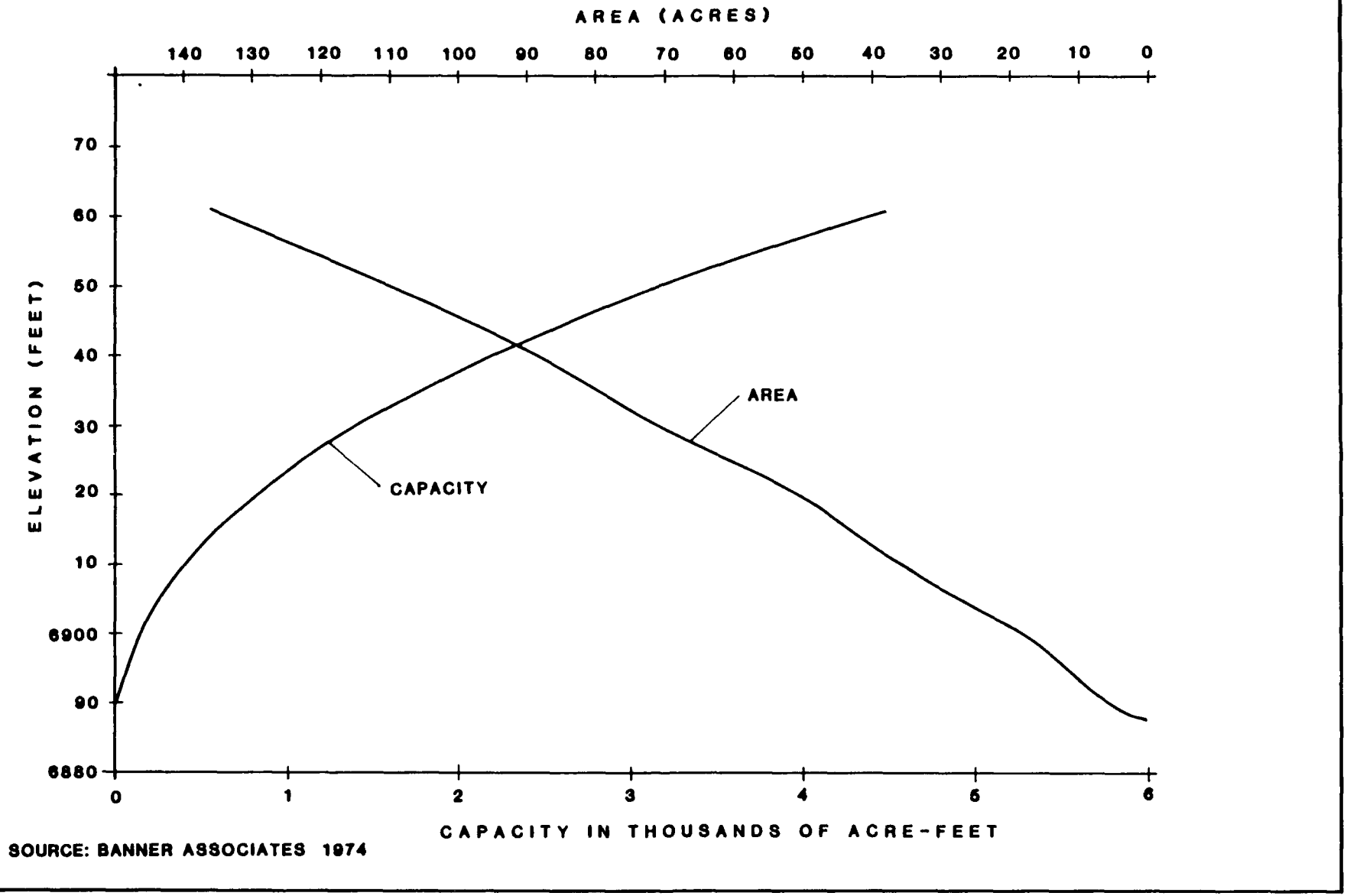
FIGURE NO. 2.6-12



SOURCE: Corps of Engineers, 1978

CRYSTAL LAKE RESERVOIR AREA AREA-CAPACITY CURVES (COE)

FIGURE NO. 2.6-13



CRYSTAL LAKE RESERVOIR AREA-CAPACITY CURVES (CBPU)

FIGURE NO. 2.6-14

to remain in operation. It is suggested that overtopping of the dam in its present condition would severely affect the stability of the dam. Remedial work would include grouting, installation of abutment drains, rock bolting, and placement of mass concrete.

On the North Fork of Crow Creek, the North Crow Creek Reservoir (also known as the New North Crow Creek Reservoir) has a maximum capacity of 1,865 acre-ft. There is an 18-inch outlet pipe and emergency spillway but no information on the capacity of either was found. The reservoir is operated similar to Granite Springs Reservoir in that the water level is lowered during the winter in preparation to capture the spring runoff. There is no current information on the safety of the dam. Figure 2.6-15 gives stage-area capacity curves for the North Crow Creek Reservoir. About 5 miles below the reservoir is the North Crow Creek Diversion with a maximum storage capacity of 98 acre-ft. Also in the North Crow system is Brush Creek Diversion which is a direct diversion from the creek with no storage capacity. High turbidity causes the diversion to be turned off intermittently.

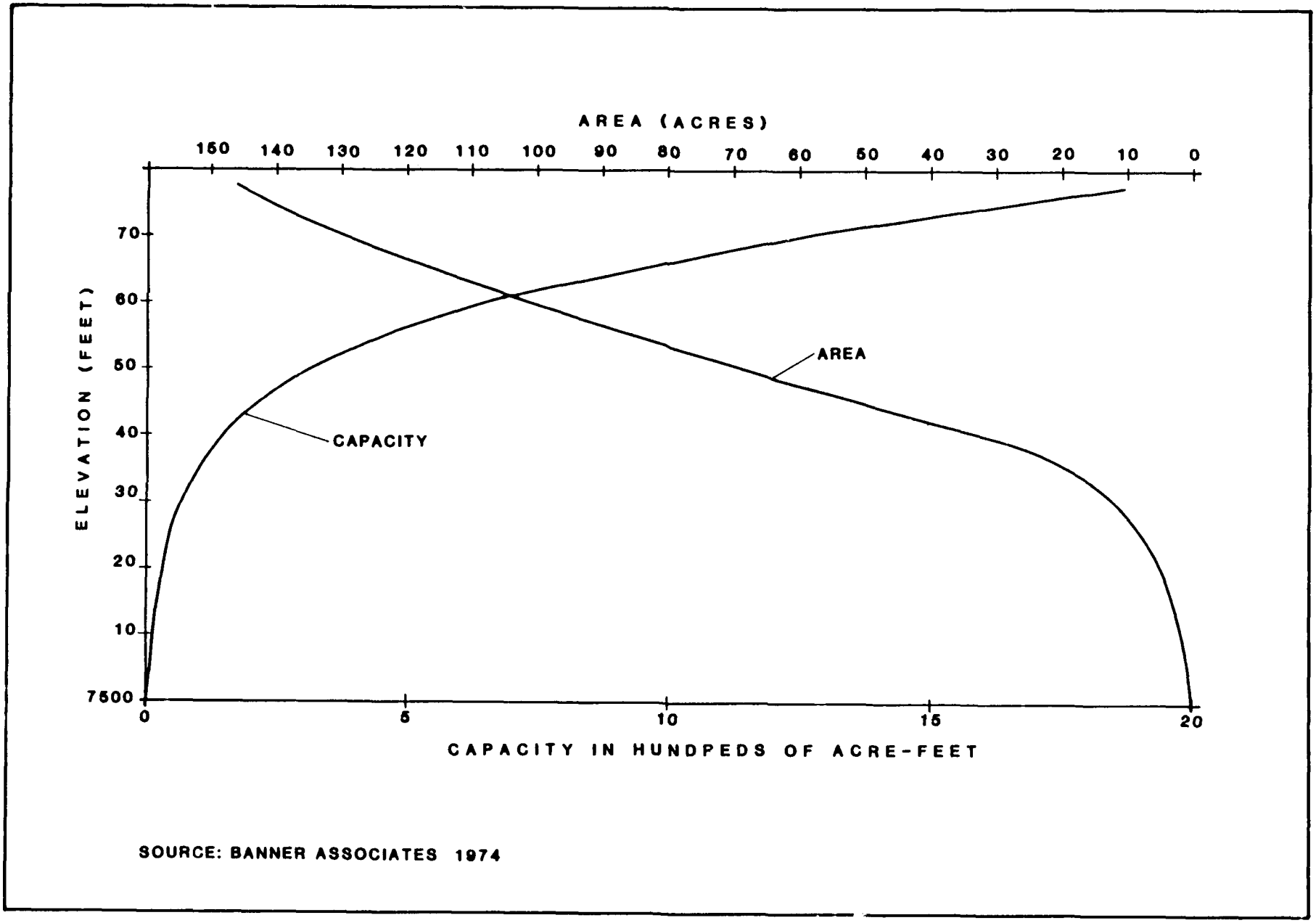
There is only one diversion on South Crow Creek and it can store only 12 acre-ft. It has a 16-inch raw water intake line and a service spillway. The dam is usually overtopped during high flows and the intake is presently not operated because of high turbidity.

There are two reservoirs below the City of Cheyenne on Crow Creek: Wyoming Hereford Ranch Reservoirs No. 1 and No. 2. They are used primarily for stock and irrigation and receive their water mainly from sewage treatment plant effluent.

2.6.3.1.3 Waste Disposal Practices

Sanitary sewage disposal techniques used in the Crow Creek watershed range from the fairly large and complex systems in Cheyenne and South Cheyenne to individual onsite septic systems. Septic tank systems may affect groundwater quality. Zero discharge lagoons may also affect the local groundwater by seepage. Systems designed to discharge effluent, e.g., the Cheyenne treatment plants, immediately affect surface water and may eventually have some effect on the groundwater quality.

The WDEQ conducted a two and one-half year study on Crow Creek to determine if it had "hydrologic and natural water quality potential to support fish" (WDEQ 1979). Over 90 percent of the flow in Crow Creek downstream of Cheyenne is wastewater treatment plant effluent during low flow conditions. The wastewater discharges have caused water quality problems of high oil and grease (sufficient to saturate benthos and preclude significant benthic life), high nutrients (sufficient to cause algal blooms in the creek and in Wyoming Hereford Ranch reservoirs), high ammonia (sufficient to be toxic to any fish), and low dissolved oxygen. Sediment samples, benthos samples, and fish studies were conducted along the stream. Some heavy metals (Cd, Cr, Zn) were found at elevated levels. The low flow conditions (generally less than 1 cfs), impact of present and past discharges, and water rights that would allow complete diversion of the stream at any of several points in the area led to the conclusion that Crow Creek should be classified as a Class IV stream requiring secondary treatment prior to discharge. Water rights downstream require discharges at present volumes to continue.



SOURCE: BANNER ASSOCIATES 1974

NORTH CROW RESERVOIR AREA-CAPACITY CURVES

FIGURE NO. 2.6-15

Individual and small area wastewater discharges in and around Cheyenne and in the rural area of Crow Creek watershed are regulated by the Cheyenne-Laramie County Health Unit. Permits have been issued for some 2,000 systems in Laramie County which is about half of the total systems in the county (Cheyenne-Laramie County Health Unit 1983). Two systems (Winchester Hills, Prairie Haven) have small lagoon facilities with the rest of the population served using individual or small group shallow trench, leach field disposal following septic tank treatment. The total population in the Cheyenne urban fringe (4,300) and in the rural Crow Creek watershed (1,500) probably discharge about 520 acre-ft/yr to the groundwater. The principal concern would be nitrate contamination since septic tank effluent contains about 20 mg/l of nitrogen; however this discharge would not likely affect surface water hydrology or quality.

2.6.3.1.4 Water Quality Conditions

Table 2.6-30 shows water quality conditions in Crow Creek throughout its length. This table is arranged so that the stations are in order, with the upper reach stations on the left.

Twelve water quality sampling points are located near Cheyenne and F.E. Warren AFB. Eight of these stations are on the base and are sampled annually for several parameters, including trace metals, pesticides, and trihalomethanes. Trace constituents, such as arsenic, barium, cadmium, chromium, chloroform, copper, cyanide, iron, lead, manganese, nickel, phenol, selenium, silver, and zinc are found in very low concentrations, if at all. Generally, these substances are identified but cannot be quantitatively measured at such low concentrations (below 100 parts per billion). Locations of the sampling stations are shown on Figure 2.6-16.

Discharges from Cheyenne area wastewater treatment facilities are required to achieve secondary treatment. Secondary treatment is defined as 30 mg/l of 5-Day Biochemical Oxygen Demand (BOD_5), 30 mg/l of total suspended solids (TSS), and effluent fecal coliform less than 200/100 ml. Data on the effluent from the three sewage treatment plants in 1980 are given in Table 2.6-31. Recent estimates for flow in 1983 indicate a combined flow of 8.58 mgd.

Table 2.6-31

SEWAGE TREATMENT FACILITIES EFFLUENT FLOW AND QUALITY- 1980

	Crow Creek STP	Dry Creek STP	SCW&SD
Flow (mgd)	3.63	2.86	0.603
BOD_5 (mg/l)	32	19	37
TSS (mg/l)	27	25	34

Source: EPA 1982.

The two CBPU treatment plants are marginally meeting secondary treatment standards. In some months, especially summer, the secondary requirements are exceeded. The SCW&SD plant does not meet secondary treatment requirements.

Table 2.6-30

CROW CREEK WATER QUALITY
(AVERAGE VALUES REPORTED)

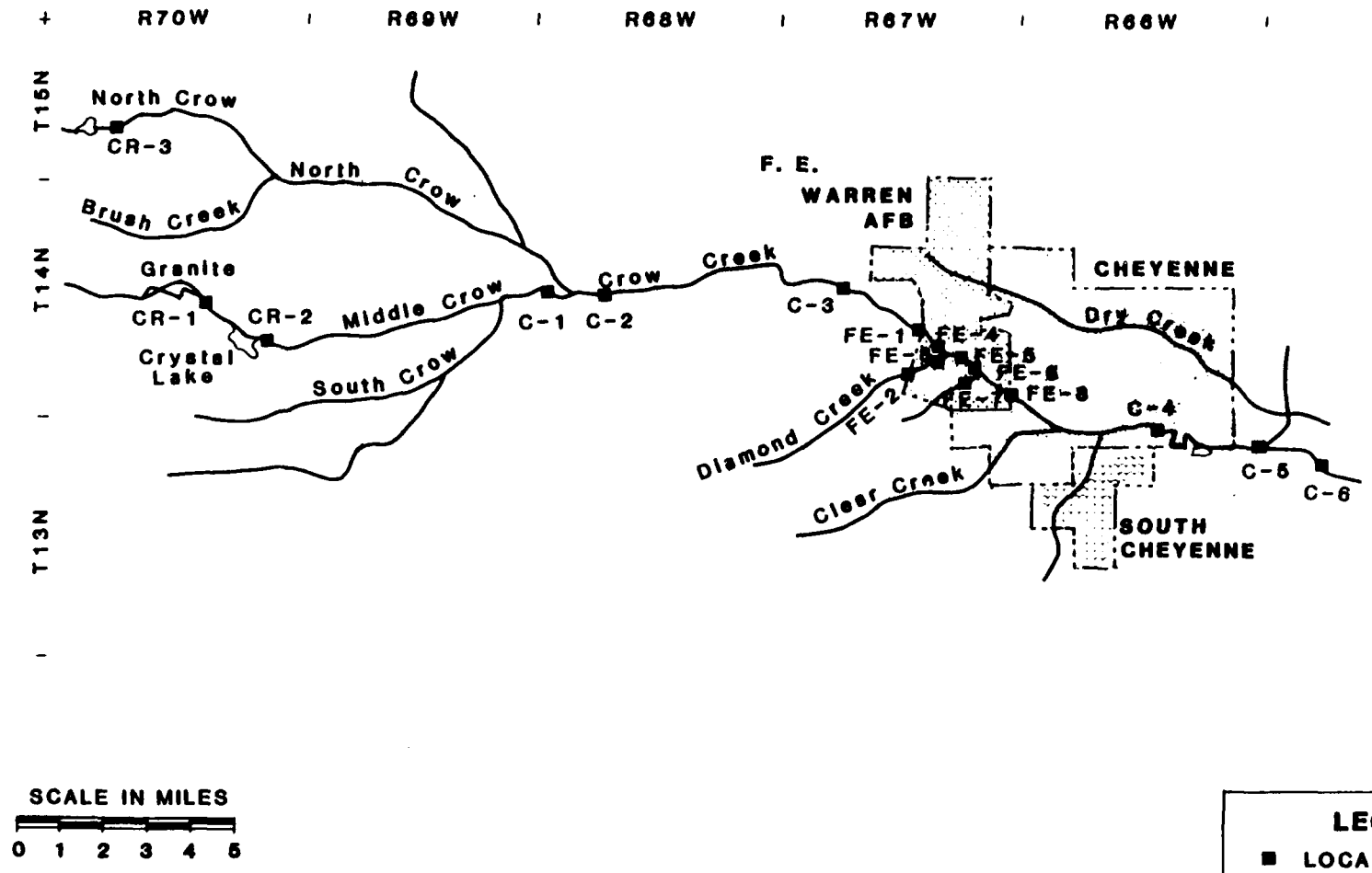
		Stations[1]																
	Units	CR1	CR2	CR3	C1	C2	C3	FE1	FE2	FE3	FE4	FE5	FE6	FE7	FE8	C4	C5	C6
Alkalinity (as $CaCO_3$)	mg/l	62.4	78	27.6	–	–	–	225	–	–	–	202	–	–	306	364	196	–
Bicarbonate (HCO_3)	mg/l	113	158	–	268	149	165	263	–	–	–	257	–	–	364	–	237	79
Carbonate (CO_3)	mg/l	–	–	–	0	0	7	–	–	–	–	–	–	–	–	–	–	39
Calcium (Ca)	mg/l	–	16	32	–	39	46	60.8	–	–	–	64	–	–	93	172	63	–
Chloride (CL)	mg/l	189	8.48	2	–	3	4	14.9	–	–	–	21	–	–	24	39	219	–
Discharge (Q)	cfs	–	–	–	1	10	20	1.0	–	–	–	–	–	–	1.0	1.0	160	2
Fluoride (F)	mg/l	–	–	–	–	1.3	1.3	1.0	–	–	–	1.0	–	–	0.9	–	–	–
Magnesium (Mg)	mg/l	–	1.0	9.7	–	6.3	7.4	12.8	–	–	–	12	–	–	18	15.2	12	–
pH	units	8.3	8.3	7.5	8	—	—	7.0	6	6	6.2	8.3	6	–	7.4	8.0	7.3	8.0
Potassium (K)	mg/l	–	–	–	–	–	–	5	–	–	–	4	–	–	5.8	–	7.9	–
SAR (Sodium Adsorption Ratio)		–	–	–	0.5	0.3	0.5	–	–	–	–	–	–	–	–	–	2.3	34
Silica (SiO_2)	mg/l	–	–	–	–	–	–	28	–	–	–	28	21	24	27	–	20	–
Sodium (Na)	mg/l	–	–	–	15	8.8	14	20	–	–	–	18	–	–	28	–	75	100
Specific Conductance	micromhos/cm	–	–	–	443	256	296	475	837	487	405	379	560	810	755	–	880	803
Sulfate (SO_4)	mg/l	–	–	13.6	17	12	18	25	–	–	–	15	–	–	23	90	81	96

Table 2.6 -30 Continued, Page 2 of 2
CROW CREEK WATER QUALITY

	Units	CR1	CR2	CR3	C1	C2	C3	FE1	FE2	FE3	FE4	FE5	FE6	FE7	FE8	C4	C5	C6
Turbidity	NTU	–	–	–	–	–	–	3	–	–	–	3.2	–	–	2.1	–	23	5
Temperature	°C	15.3	18.0	18.3	–	–	–	14	10	13	19	28	12	–	15.8	12.8	12.8	12.7
BOD	mg/l	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
COD	mg/l	–	–	–	–	–	–	11	64	15	22	20	10	15	14	–	131	60
Hardness (as $CaCO_3$)	mg/l	120	74.5	–	205	123	145	244	–	–	–	222	–	–	318	–	206	464
Dissolved Oxygen	mg/l	7.4	7.0	7.5	–	–	–	12	–	–	–	11	–	–	108	9.9	2.6	6.1
TDS	mg/l	189	200	96	289	–	–	382	578	325	293	278	363	511	782	572	475	610
Nitrate	mg/l	–	–	–	–	0.2	–	0.49	0.3	0.1	0.1	0.88	1.1	0.2	0.8	0.18	–	0.89
Ammonia	mg/l	–	–	–	–	–	–	0.17	0.3	0.2	0.2	0.1	0.2	0.2	0.41	0.5	1301	–
Phosphorus	mg/l	–	–	–	–	–	–	0.1	0.2	0.1	0.1	0.2	0.1	0.3	0.18	–	5.8	1.8
Boron	µg/l	–	–	–	–	0.02	–	1	–	–	–	–	–	–	0.4	–	–	–
Iron	µg/l	–	–	1100	–	–	–	219	–	–	320	100	100	600	203	–	452	–
Arsenic	µg/l	–	–	–	–	–	–	10	–	–	–	10	–	–	15	–	8	–
Copper	µg/l	–	–	188	–	–	–	59	34	20	26	58	30	30	58	–	38	–
Fecal Coliform	#/100 ml	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	$5x10^6$	–

Note: 1 Station locations are shown on Figure 2.6- 16.

Source: USGS WATSTOR Data Program 1983.



LOCATIONS OF WATER QUALITY SAMPLING STATIONS IN CROW CREEK WATERSHED

FIGURE NO. 2.6-16

Except for the influence of the Cheyenne Urban Area, Crow Creek water quality conditions are generally good.

Average existing water erosion and sediment delivery for Crow Creek in the ROI are shown in Table 2.6-32. A total erosion of 1,210,000 tons per year (T/yr) is estimated, with sediment delivery to Crow Creek estimated to be 205,000 T/yr. These values were calculated using procedures described in Section 2.6.3 and provide only a gross estimate of the existing situation.

Table 2.6-32

EROSION AND SEDIMENTATION
CROW CREEK WATERSHED

	Erosion (1,000 T/yr)				Sedimentation	
Area	Rangeland	Cropland	Other	Total	Ratio	Delivery 1,000 T/yr
Upstream of Cheyenne	667	20	42	729	0.163	119
Between Cheyenne and Carpenter	445	30	7	481	0.178	86

2.6.3.2 Other Watersheds in the Region of Influence Potentially Impacted by Project Construction

2.6.3.2.1 Lodgepole Creek Watershed

Lodgepole Creek watershed has a length of about 180 miles and a drainage area of about 3,250 sq mi upstream of its confluence with the South Platte River.

The Laramie Range of the Southern Rocky Mountains forms the western edge of this watershed. The area east of the mountains is within the High Plains segment of the Great Plains Physiographic Province. In this area, Lodgepole Creek is in a steep-walled, flat-bottomed valley cut into the generally smooth eastward sloping surface of the plains averaging one and one-half miles in width and 100 to 150 feet in depth.

The generally smooth upland surface of the Great Plains portion of the watershed is interrupted by two prominent escarpments, each about 250 feet high, located near Islay and Pine Bluffs, respectively.

Tributaries to Lodgepole Creek within the ROI include the north and south forks of Lodgepole Creek, Nine Mile Draw, Antelope Draw, Chivington Draw, Spring Creek, Muddy Creek, and Sidney Draw. All are intermittent streams.

Incorporated communities within the Lodgepole Creek ACS include Albin, Pine Bluffs, and Burns in Laramie County, Wyoming, and Bushnell, Kimball, and Dix in Kimball County, Nebraska.

2.6.3.2.1.1 Streamflow

As of 1981, the maximum discharge of record on Lodgepole Creek at Bushnell, Nebraska of 16,500 cfs occurred on September 15, 1950, and the minimum daily discharge of record of 0.09 cfs occurred on July 20, 1976. The average annual discharge at Lodgepole Creek at Bushnell, Nebraska is 12.3 cfs based on 39 years of record between October 1931 and September 1970. Recent data indicate the average flow has decreased to 5 cfs due to increased groundwater withdrawals upstream of Bushnell, Nebraska. The USGS calculated potential flood flows for various return periods by Log-Pearson analysis for Lodgepole Creek near Bushnell, Nebraska with values as given below:

	Q_2	Q_5	Q_{10}	Q_{25}	Q_{50}	Q_{100}
Lodgepole	210 cfs	970 cfs	2,330 cfs	6,270 cfs	12,300 cfs	22,900 cfs

The flow (Q) values are shown for various return periods i.e. Q_{10} is the flow expected for a 10-year return period.

2.6.3.2.1.2 Lakes and Reservoirs

The principal lakes and reservoirs within the Lodgepole Creek Basin are Oliver Reservoir (7,480 acre-ft) located east of Bushnell, Nebraska, Bennett Reservoir (970 acre-ft) located northeast of Kimball, Nebraska, and One Mile or Pole Creek Reservoir (3,392 acre-ft) located in central Laramie County. Oliver Reservoir was constructed in 1911 as a major feature of the Kimball irrigation project. Storage capacity restrictions were placed on Oliver Reservoir by the COE in the 1970s due to safety considerations and since the local irrigation company was dissolved. In recent years Oliver Reservoir has been revitalized as a recreational reservoir. Bennett Reservoir is a small reservoir which was built in the late 1920s and has since partly filled with sediment.

2.6.3.2.1.3 Waste Disposal Practices

Sanitary sewage treatment techniques used in the Lodgepole Creek Basin range from single cell lagoon evaporation ponds to septic tanks. The septic systems could potentially affect the groundwater quality. The zero discharge lagoons probably seep effluent to the groundwater in some locations.

The WDEQ studied Lodgepole Creek in its statewide water quality assessment. It found that no potential nonpoint source polluters existed in the area studied, from the south fork tributary to below Highway 85. Also, there were no point source discharges except below Highway 85. In either case, point source or nonpoint source, no documented problems exist.

Wastewater in the Lodgepole Creek watershed derives primarily from municipal sewage and minor sewage generated at the LCFs and discharges of sump water pumped out at the LFs. Communities in the watershed include Burns, Pine Bluffs, Bushnell, Kimball, and Dix. Three LCFs and 33 LFs are located in the watershed.

The communities of Burns, Pine Bluffs, and Dix operate zero discharge treatment lagoons which do not release effluent to any streams or channels. The Burns lagoon has a design capacity of 0.03 mgd and a current flow of 0.02 mgd. In Pine Bluffs, the current flow of 0.10 mgd exceeds the design capacity of 0.09 mgd. Pine Bluffs is contemplating enlargement of the facility. The current flow to the Dix lagoon of 0.04 mgd also exceeds the design capacity of 0.03 mgd.

Bushnell and Kimball operate treatment facilities which discharge to Lodgepole Creek. The Bushnell facility provides primary treatment with a lagoon, and currently operates at the design capacity of 0.02 mgd. The Kimball facility consists of an oxidation ditch with a design capacity of 0.58 mgd, and the current flow is 0.30 mgd.

Two of the three LCFs have a septic tank/sewage lagoon combination for treating wastes. Correspondence from F.E. Warren AFB indicates that some problems exist with these systems, and they are currently being investigated (Deputy Base Civil Engineer). The third LCF has a zero discharge lagoon with a capacity of 9,000 cubic feet (cf), which operates satisfactorily.

No toilet facilities are provided at LFs. Maintenance crews take small portable units with them if they will be in the field for a long period. Sumps are located in the silos, and water pumped from these sumps is discharged at the surface over a gravel area and infiltrates to the soil. The sump accumulates 40 to 80 gallons before the pump is automatically operated. It is estimated that the sump is emptied on an average of twice monthly. Normal maintenance includes running 110 to 130 gallons of clean water through the sump once annually, and a complete cleaning of the sump every 3 years.

The sump discharge contains varying amounts of bacteria, depending on the frequency of pumping. In drier sites, these bacteria have been concentrated enough to produce ammonia in quantities sufficient to activate a hydrazine leak detection system, leading to an average of 25 false alarms per year. Types of material that could appear in the sump water are a function of spills during maintenance and are not quantifiable except that probably less than 3 gallons of material are ever spilled. Materials include coolants (glycol, sodium chromate), and oil and grease. Sodium chromate is classified as a hazardous waste by the EPA.

In the case of a hazardous waste spill at an LF or LCF, Operations Plan 423-83 concerning spill prevention, control, and countermeasures, and Wyoming regulations describe the actions to be taken. The Operations Plan includes a five-phase response effort: discovery and notification, containment and countermeasures, cleanup and disposal, restoration, and recovery of damages and enforcement.

2.6.3.2.1.4 Water Quality Conditions

Base flow surface water quality on Lodgepole Creek is similar to groundwater quality due to the interrelated nature of surface and subsurface flows. Surface water quality, TDS, specific conductance, and hardness data indicate that mineralization increases downstream. While this does not account for variations associated with long-term changes, the general observation that the spatial distribution of water quality is from high quality in the upper reaches to lower quality in the downstream reaches is apparent.

Water quality conditions are generally good in most reaches of Lodgepole Creek and its tributaries, except where locally influenced by agricultural practices.

Water from the upper Lodgepole Creek drainage basin in Wyoming has a maximum specific conductance of 569 micromhos per centimeter, maximum sodium content of 60 percent, and a maximum sodium adsorption ratio of 3.0. The specific conductance is slightly higher in the lower portion of the basin in Nebraska. Groundwater and surface water of the upper Lodgepole Creek drainage basin within Laramie County, are considered suitable for irrigation. Data show that dissolved solids have peak concentrations from June through August (STORET 1983).

Average water erosion and sediment delivery for Lodgepole Creek in the ROI are shown in Table 2.6-33. A total erosion of 5,590,000 T/yr is estimated with 730,000 T/yr reaching the stream.

2.6.3.2.2 Horse Creek Watershed

Horse Creek originates in the Laramie Mountains east of Laramie, Wyoming and flows generally eastward to the Wyoming-Nebraska state line where it veers sharply to the north to its confluence with the North Platte River about 4.5 miles east of the Wyoming-Nebraska state line. Horse Creek drains approximately 1,570 sq mi of which only a small part is located within Nebraska. It is approximately 136 miles long. A considerable number of diversions have been made from Horse Creek in Wyoming. Streamflow on Horse Creek is affected by diversions, spills, and return flow.

Table 2.6-33

EROSION AND SEDIMENTATION
LODGEPOLE CREEK WATERSHED

	Erosion (1,000 T/yr)				Sedimentation	
Area	Rangeland	Cropland	Other	Total	Ratio	Delivery 1,000 T/yr
Upstream of Oliver Reservoir	806	1,684	21	2,510	0.123	310
Between Oliver Reservoir and Sidney Draw	403	916	-	1,320	0.139	180
Sidney Draw	750	1,005	-	1,760	0.133	240

2.6.3.2.2.1 Streamflow

The historic average annual streamflow on Horse Creek near Lyman, Nebraska is 68.6 cfs based on 50 years of record between February 1931 and September 1981. As of 1981 the maximum discharge of record on Horse Creek near Lyman of 5,110 cfs occurred on June 6, 1967, and the minimum daily discharge of record of 0.4 cfs occurred on February 1 and 2 of 1949.

2.6.3.2.2.2 Lakes and Reservoirs

Major lakes and reservoirs in the Horse Creek drainage area include Arnold (1,140 acre-ft), Goshen Hole (4,950 acre-ft), Sinnard (1,540 acre-ft), Hawk Springs (16,700 acre-ft), Glomill (2,230 acre-ft), and Goshen No. 2 (890 acre-ft).

Hydrologic characteristics and water quality conditions for lakes, ponds, and reservoirs are generally not available.

2.6.3.2.2.3 Waste Disposal Practices

Wastewater in the Horse Creek watershed derives primarily from municipal sewage, sewage generated at LCFs, and sump water discharged at the LFs. Communities within the ROI in the Horse Creek watershed include La Grange and Yoder. Four LCFs and 33 LFs are located in the Horse Creek watershed.

La Grange operates a new sewage treatment lagoon with zero discharge. Current flow is estimated at 0.02 mgd.

The Yoder facility is a zero discharge lagoon with a design capacity of 0.04 mgd. The current flow is 0.03 mgd.

Waste disposal practices at LCFs and LFs have been described in Section 2.6.3.2.1.3 (Lodgepole Creek). Two LCFs in the Horse Creek watershed have septic tank/lagoon combinations and two have zero discharge lagoons. Spill management practices are as described in Section 2.6.3.2.1.3.

As in the other basins outside the Crow Creek Basin, the two major waste disposal practices used are the septic tank in rural areas and the no-discharge lagoon. Both systems pose potential adverse effects to the groundwater in the area. Seepage to a high groundwater table is common among no discharge lagoons. The communities in the basin all have no discharge lagoons. La Grange was documented to be a point source for fecal coliform in the Horse Creek Basin in 1980. Since then a new lagoon has been built and there has been no update on the pollution.

There are no documented nonpoint source pollution problems on Horse Creek.

2.6.3.2.2.4 Water Quality Conditions

Water quality is generally good in the Horse Creek watershed. Specific conductance values at the various water quality sampling stations (Figure 2.6-7) range from 450 to 900 micromhos per centimeter with a typical value of about 600. Sodium adsorption ratios vary from 0.5 to 1.7 with sodium ranging from 14 to 57 mg/l, calcium from 22 to 118 mg/l and magnesium from 10 to 30 mg/l (STORET 1983).

Erosion and sedimentation in the watershed are estimated in Table 2.6-34.

2.6.3.2.3 Pumpkin Creek Watershed

Pumpkin Creek, also known as Pumpkin Seed Creek, originates in the western portion of Banner County, Nebraska and flows generally eastward to its confluence with the North Platte River east of Bridgeport, Nebraska. The Pumpkin Creek watershed encompasses approximately 1,020 sq mi and is about 72 miles long.

Table 2.6-34

EROSION AND SEDIMENTATION HORSE CREEK WATERSHED

	Erosion (100 T/yr)				Sedimentation	
Area	Rangeland	Cropland	Other	Total	Delivery Ratio	1,000 T/yr
Upstream of Hawk Springs	1,557	148	35	1,740	0.137	238
Hawk Springs to North Platte	834	1,418	28	2,280	0.125	285

2.6.3.2.3.1 Streamflow

Except for about 48 sq mi in the southwest corner and about 35 sq mi in the northeast corner, all of Banner County is drained by Pumpkin Creek and its principal tributary, Lawrence Fork, which empties into Pumpkin Creek about 7 miles east of the east county line. Pumpkin Creek enters Banner County at an altitude of about 4,440 feet and leaves at an altitude of about 3,490 feet. The average gradient of its floodplain in Banner County is about 20 ft/mi.

Pumpkin Creek probably has surface flows most of the year below Section 32, T20N, R56W, its flow in dry weather being maintained by seepage of groundwater from adjacent sediments. All its tributaries within the county are ephemeral, flowing only in response to runoff-producing storms. Some of the runoff collects in depressions on the lowland surface and thus never reaches Pumpkin Creek.

The mean flow of Pumpkin Creek at Bridgeport, Nebraska is 32.1 cfs based on 39 years of record from 1931 to 1970.

2.6.3.2.3.2 Lakes and Reservoirs

There are no large water storage reservoirs within the Pumpkin Creek watershed, however there are numerous stockponds and dugouts.

2.6.3.2.3.3 Waste Disposal Practices

There are no communities with wastewater facilities in the Pumpkin Creek watershed. Two LCFs and 17 LFs are located within the watershed. The facilities at the LCFs are lagoon type as described in Section 2.6.3.2.1.3. The description of waste disposal practices at those facilities and LFs also applies to facilities in the Pumpkin Creek watershed.

2.6.3.2.3.4 Water Quality Conditions

Water quality is generally good in the Pumpkin Creek watershed. TDS values range from 200 to 380 mg/l, calcium from 32 to 59 mg/l, magnesium from 9 to 14 mg/l, and sodium from 10 to 65 mg/l.

The water is generally a calcium bicarbonate type that is suitable for all uses. There are no apparent trends in the data, although it appears that most of the tributaries are slightly less mineralized (as indicated in TDS) than Pumpkin Creek itself.

No data were available to indicate seasonal variations in Pumpkin Creek.

An estimated 2,365,000 T/yr erosion occurs in the watershed with about 298,000 T/yr reaching streams.

2.6.3.2.4 Chugwater Creek Watershed

The only perennial stream tributary to the Laramie River within the water resources ACSs is Chugwater Creek which drains approximately 680 sq mi of which approximately 75 sq mi are within Laramie County, Wyoming with the remainder within Platte County, Wyoming. Chugwater Creek is a small stream which meanders within a moderately wide floodplain ranging from 1,000 to 2,000 feet in width. Streamflows on Chugwater Creek are affected by numerous irrigation diversions.

2.6.3.2.4.1 Streamflow

The historic mean annual outflow of Chugwater Creek near Uva, Wyoming averaged 34.5 cfs based on 3 years of record (October 1965 - September 1968). Average flow near Wheatland is about 27 cfs.

2.6.3.2.4.2 Lakes and Reservoirs

There are no large water storage reservoirs within the Chugwater Creek Basin. Numerous stockponds and dugouts do exist however.

2.6.3.2.4.3 Waste Disposal Practices

Wastewater in the Chugwater Creek watershed derives from municipal sewage, industrial discharges, and sewage generated at LCFs. No information is available concerning industrial discharges, although water quality problems related to industrial discharge have been noted near Wheatland. Communities with wastewater facilities in the Chugwater Creek drainage include Chugwater and Wheatland. There are 13 LFs and 1 LCF in the watershed.

Chugwater currently operates a sewage lagoon with discharge to an irrigated meadow. The lagoon currently operates at the design capacity of 0.03 mgd. Wheatland operates a sewage lagoon with current flows of 0.30 mgd. Discharge is to Rock Creek which feeds the Laramie River.

Waste disposal practices for the LCF and LFs are described in Section 2.6.3.2.1.3. The LCF has a septic tank/lagoon type system.

2.6.3.2.4.4 Water Quality Conditions

The water in Chugwater Creek has calcium and magnesium cations and sulfate and and bicarbonate anions. The TDS is above the 500 mg/l EPA secondary standard for drinking water. It is, however, suitable for other uses such as stock watering, irrigation, or industrial purposes. Also, the water is classified as hard.

Insufficient data exist to illustrate spatial variations in water quality.

Existing erosion in the watershed is estimated at 1,768,000 T/yr with about 240,000 T/yr reaching the streams.

2.6.3.3 Other Watersheds in the Region of Influence

Surface water hydrology and quality are not expected to be significantly changed over existing conditions in drainages outside of those discussed in Section 2.6.3.2 as a result of the project. The following information provides an overview of hydrology and water quality conditions existing in these watersheds.

2.6.3.3.1 North Platte River

2.6.3.3.1.1 Hydrology

The North Platte River is approximately 665 miles long and drains approximately 34,900 sq mi between its source in north-central Colorado and its confluence with the South Platte River. Elevations within the North Platte Basin range from about 13,000 feet to 2,800 feet above mean sea level.

Streamflow in the North Platte Basin is derived from precipitation, largely snowmelt, and groundwater seepage, which includes underflow from surrounding basins and irrigation return flows from both surface water diversions and pumped groundwater.

A U.S. Supreme Court decree of 1945 apportioned the natural flow of the North Platte River between Colorado, Wyoming, and Nebraska. Water is diverted from the North Platte Basin in Colorado to the South Platte Basin. Under the terms of the U.S. Supreme Court decree, Colorado is limited to exporting no more than 60,000 acre-ft of water from the North Platte Basin in any 10-year period.

Reservoirs and irrigation projects within the North Platte Basin significantly affect streamflow on the North Platte River. The U.S. Bureau of Reclamation has six major reservoirs on the North Platte River in Wyoming; Pathfinder, Guernsey, Greys Reef, Seminoe, Alcova, and Glendo. Dates of construction, capacities, and purposes of these reservoirs are presented in Table 2.6-35.

Table 2.6-35

U.S. BUREAU OF RECLAMATION
RESERVOIRS ON THE NORTH PLATTE RIVER

Reservoir	Year Of Construction	Capacity (acre-ft)	Purpose
Seminoe	1939	1,011,000	Storage
Pathfinder	1909	1,016,000	Storage
Alcova	1938	190,700	Storage
Glendo	1957	796,000	Flood Control Hydroelectric Storage
Grays Reef	1961	1,780	Streamflow Regulation
Guernsey	1927	45,300	Storage Hydroelectric Streamflow Regulation

These reservoirs moderate streamflows on the North Platte River, reducing peak streamflows by storing spring snowmelt for use later in the year and increasing low flows through subsequent reservoir releases for irrigation and hydroelectric power generation.

Releases from reservoirs and diversions to and returns from the irrigation systems influence flow of the North Platte.

Flow for the North Platte at the Wyoming-Nebraska state line for 1958 to 1973 averaged 772 cfs.

The 7-day, 10-year low flow is 80 cfs and the 1-day, 50-year high flow is 12,800 cfs at this location. Average flow at Bridgeport is 1,270 cfs.

Both peak flows and annual mean flows have decreased on the North Platte River since 1895 due to the reservoir and irrigation project developments. There has, however, been no significant change in annual mean flows on the North Platte River since 1935. Reduced flows on the North Platte River have resulted in the North Platte River becoming narrower.

Flooding on drainageways tributary to the North Platte River can result from snowmelt or heavy rainfall or a combination thereof. Although flood events on the North Platte River are generally slow to peak and of long duration, drainages tributary to the North Platte River are prone to flooding as a result of heavy rainfall which causes local flooding of short duration with little warning time.

2.6.3.3.1.2 Water Quality

The North Platte system is the largest and most complex in the ROI. For the purposes of the analysis of water quality the system has been subdivided into three parts, the river and its tributaries, the extensive system of canals, laterals and drains, and the larger reservoirs in the ROI.

The distinction between tributaries and natural drains is sometimes difficult to make. Drainages that in past years have shown no base flow but now flow year-round are generally those affected by irrigation return flow or have base flow as a result of the water table rising due to groundwater applications. These have been considered part of the irrigation system in this section.

The river itself has many water quality sampling stations located along its length. Water quality degrades as the river proceeds through the ROI as indicated by a 74-percent increase in TDS from the station in Converse County, Wyoming to the station in Garden County, Nebraska. Nitrate levels increase from approximately 0.5 mg/l to 2.5 mg/l, a 5-fold increase. It is likely that this is due to the agricultural uses for water in the North Platte Basin. The general water quality is poor because of the high TDS concentrations. It is suitable for irrigation, industrial, and stock watering uses but because the TDS values exceed 500 parts per million (ppm), which is the EPA secondary drinking water standard, it is not desirable for potable use.

The surface water quality in the North Platte River is affected by several man-caused factors. Irrigation appears to be the primary influence; however, waste discharges, industrial uses, and reservoir operations also influence

water quality. Reservoirs in the ROI are typically used to store water for irrigation. Sanitary wastewater discharges into the North Platte River were approximately 6.5 mgd (10 cfs) in 1982. The flow in the North Platte River near the eastern edge of the ROI for the same period was 1,185 cfs average. Thus, treated wastewater is less than 1 percent of the total flow. There are few industries along the North Platte in the ROI that discharge; therefore, it is assumed that industrial wastewaters constitute a very small portion of the flow.

Reservoirs can act as sinks or sources for different pollutants. Suspended matter may be settled out in a reservoir but dissolved solid concentrations may be increased because of the water losses from evaporation. Lower strata of lakes may have very little or no dissolved oxygen. However, once discharged from a reservoir, natural aeration will occur and oxygen will be replenished. Reservoirs may also cause changes to downstream erosion and sedimentation patterns.

The two major reservoirs in the ROI are Glendo and Guernsey reservoirs. Table 2.6-36 shows water quality conditions for these two reservoirs. Data for Guernsey Reservoir are actually from a stream sample site immediately below the reservoir.

The most significant factor contributing to the changes in surface water quality in the North Platte River is irrigation return flow.

The average quality of the water in irrigation drains compared with average value of water quality parameters from the diversion canals show that the return water has a higher conductivity in nearly all cases. Also, nitrate levels in the drains are approximately ten times higher than those in the canals.

The quality of the return water is slightly affected by geologic conditions, with water from the south side of the river slightly more mineralized (higher bicarbonate, sulfate, and chloride) but lower in silica. This may also be affected by the source of the irrigation water. The large increase in nitrate is an indication of the affect of fertilizer application.

Wastewater on the North Platte River derives from municipal sewage and industrial discharge. No information is available on industrial discharges. Municipal treatment facilities exist for the communities of Fort Laramie, Lingle, Torrington, Henry, Morrill, Mitchell, Scottsbluff, and Terrytown. No LCFs or LFs are located in the watershed, except in Horse, Pumpkin, and Chugwater Creek watersheds, which all drain into the North Platte.

Fort Laramie, Torrington, and Mitchell operate zero discharge lagoon systems. The Fort Laramie system has a design capacity of 0.05 mgd and 0.02 mgd current flow. The Torrington system has a current flow of 0.55 mgd, with a 1.2 mgd design capacity. The Mitchell system has a current flow of 0.47 mgd, in excess of the 0.24 mgd design capacity.

Lingle operates a lagoon of 0.05 mgd design capacity, with 0.04 mgd current flow. Henry operates a lagoon at the design capacity of 0.01 mgd. Morrill operates a lagoon with 0.14 mgd design capacity at current flows of 0.31 mgd. Scottsbluff operates an aerated lagoon with a design capacity of

Table 2.6-36

WATER QUALITY CONDITIONS OF NORTH PLATTE RESERVOIRS

	Units	Glendo			Guernsey		
		Minimum	Average	Maximum	Minimum	Average	Maximum
Temperature	°C	15	21	26	0.0	11.5	23
Conductivity	Micromhos	490	623	680	425	760	1,390
Dissolved Oxygen	mg/l	7.0	8.8	13.0	11.6	12.9	14.4
pH	units	8.1	8.4	8.6	7.1	7.9	8.7
Alkalinity (as $CaCO_3$)	mg/l	131	137	143	107	167	260
Phosphorus	mg/l	0.012	0.05	0.081	0.0	0.062	1.94
Ammonia & Ammonium	mg/l	0.111	0.14	0.169	0.02	0.077	0.12
TDS	mg/l	342	374	420	277	527	1,010

Note: The two reservoirs are similar in quality; however, the conductivity and TDS in Guernsey Reservoir, the downstream reservoir, are higher. The higher dissolved oxygen in this reservoir may be due to the fact that the samples are taken from below the reservoir where some aeration may have taken place.

Source: WATSTOR 1983.

3.1 mgd and current flows of 2.5 mgd. Terrytown operates an aerated lagoon with a design capacity of 0.21 mgd and current flows of 0.24 mgd.

Four Launch Facilities are located in the Goshen Hole area that drains about 560 sq mi to the North Platte between the confluence of the Laramie River and the confluence of Horse Creek with the North Platte River. Average flow of the North Platte in this area is about 780 cfs with flow primarily controlled by Wheatland, Glendo, and Guernsey reservoirs. Wastewater discharges include Scottsbluff (2.6 mgd) and Terrytown (0.24 mgd). Water quality is typified by high total dissolved solids that increases significantly in the downstream direction as a result of irrigation return flow. Existing erosion in the 560 sq mi area is estimated at 1,200,000 T/yr with 210,000 T/yr reaching streams or irrigation ditches in the area.

2.6.3.3.2 Laramie River

2.6.3.3.2.1 Hydrology

The Laramie River is the major tributary to the North Platte River within the water resources ROI. The Laramie River originates in north-central Colorado and drains to the north and the northeast to the North Platte River. The Laramie River drains a total of approximately 4,600 sq mi of which approximately 294 sq mi are located in Colorado and the remainder are located within Wyoming.

A U.S. Supreme Court decree permits Colorado to divert 49,375 acre-ft of water from the Laramie River for use in Colorado during each calendar year with the remainder and return flows apportioned to Wyoming (Southeastern Wyoming Resource Conservation and Development Area Plan 1982).

The average annual discharge of the Laramie River at Laramie, Wyoming is approximately 102 cfs based on 19 years of record between 1951 and 1970. As of 1970 the maximum discharge of record on the Laramie River of 3,250 cfs occurred on June 15, 1957, and the minimum daily flow of record of 0.1 cfs occurred on September 7 through 9, 1950 (USGS 1973).

The Wheatland Irrigation District stores and diverts water from the Laramie River to irrigate approximately 53,000 acres. Major reservoirs on the Laramie River include Wheatland Reservoir No. 1 (9,370 acre-ft), Wheatland Reservoir No.2 (98,924 acre-ft), Wheatland Reservoir No. 3 (90,000 acre-ft) and Greyrocks Reservoir (104,000 acre-ft).

The historic mean annual outflow of the Laramie River above its confluence with the North Platte River was approximately 82,000 acre-ft/yr prior to construction of Greyrocks Dam and Reservoir in 1980 (Dames and Moore 1980).

Present streamflows on the Laramie River are expected to be less than historic (pre-1980) due to 1) construction and operation of both Greyrocks Dam and Reservoir and Johnson wellfield to supply water to the Wheatland generating station of the Missouri Basin Power Project; 2) planned construction and operation of Corn Creek irrigation project in Wyoming; and 3) increased groundwater pumping in Nebraska to support irrigated agriculture.

Greyrocks Dam and Reservoir on the Laramie River in Platte County, Wyoming is expected to deplete the Laramie River by about 26,000 acre-ft/yr on the average. Under the terms of an agreement (December 1978) between the State of Nebraska, the Rural Electrification Administration, the COE, the Missouri Basin Power Project, etc., the Greyrocks Dam and Reservoir on the Laramie River is to augment historic flows on the North Platte River during the months of July, August, and September, provided Greyrocks Reservoir contains at least 50,000 acre-ft of water in storage.

The North Laramie River is a left bank tributary to the Laramie River encompassing approximately 530 sq mi. The average annual flow of the North Laramie River near Wheatland, Wyoming (drainage area = 370 sq mi) is 38.8 cfs based on 40 years of record between October 1914 and September 1970. There are 7 small reservoirs upstream of the North Laramie River gaging station near Wheatland, Wyoming having a total storage capacity of about 290 acre-ft. Irrigation diversions on the North Laramie River above the gaging station near Wheatland irrigate approximately 4,800 acres.

As of 1970 the maximum discharge of record on the North Laramie River of 9,260 cfs occurred on July 27, 1951 and periods of no flow occurred in 1915, 1917, 1954, 1956, and 1960 (USGS 1973).

There are no incorporated communities within the North Laramie River Basin.

2.6.3.3.2.2 Water Quality

Water quality conditions in the Laramie River and its tributaries indicate that the water is principally a calcium bicarbonate type that is suitable for most uses, although TDS is near 500 mg/l. The TDS and conductivity in the Laramie River does not vary considerably with TDS being approximately 520 mg/l.

2.6.4 Groundwater Hydrology and Quality

The development of new wells or use of groundwater from existing wells to meet project-related water demands could affect existing groundwater users. In order to be able to evaluate this, an inventory of existing rights and wells was developed. Table 2.6-37 summarizes well locations and production levels near Cheyenne. Table 2.6-38 summarizes the number and production levels of wells near municipalities in other ACSs. Table 2.6-39 documents groundwater rights within 1 mile of LFs and LCFs.

2.6.4.1 Hydrogeologic Units

The stratigraphy of the ACS consists of a thick sequence of sedimentary units ranging in age from Cambrian to Recent, which overlie the Precambrian crystalline basement rocks. Although the hydrologic properties of these units vary considerably, several groupings can be made on the basis of water-bearing and transmitting characteristics. Consequently, the numerous geologic units present in the ROI have been grouped into eight major hydrogeologic units for the purpose of describing the groundwater system (Table 2.6-40).

The eight hydrogeologic units are, from oldest to youngest: Precambrian crystalline basement rocks, Pennsylvanian-Permian Casper Formation (equivalent

Table 2.6-37

CHEYENNE AREA WELLS

	T13N, R66W		T13N, R67W		T14N, R67W		T14N, R66W	
Section	<100gpm	>100gpm	<100gpm	>100gpm	<100gpm	>100gpm	<100gpm	>100gpm
1	1		13		39		0	
2	2		1	3	4		3	
3	-	2	0		2		2	
4	22		0		16		27	
5	18	3	5		0		29	1
6	5		12		1		85	
7	1		2		2	2	97	
8	36		1		2		80	
9	14	1	0		0		40	
10	1		0		2		64	
11	-		7		0		3	
12	3		18	2	69		2	
13	1		23	13	90	1	3	
14	0		4	4	2		37	
15	0		1		0		123	
16	3		19	2	0		11	
17	17		8	2	0		40	
18	18	4	4	3	6	2	76	
19	0		2		2		56	
20	7		0		1		6	
21	1		1		0		18	
22	1		2		0		89	
23	1		3		0	3	26	
24	0		0		18	1	8	
25	1		0		33		40	
26	0		1		0		109	
27	0		1		5		174	
28	0		3		49		3	
29	6		1		0		15	
30	14		0		2		3	
31	45	2	0		2		13	
32	24		0		2		10	
33	0		1	1	39		19	
34	1		2		30		10	
35	1		0		4		2	
36	1		1		27		21	

Table 2.6-38

WELLS NEAR COMMUNITIES IN OTHER AREAS OF CONCENTRATED STUDY

County City	Wells <1 mi <100gpm	Wells <1 mi >100gpm	Wells 1-2 mi <100gpm	Wells 1-2 mi ≥100gpm	Municipal <2 mi Number	Municipal <2 mi gpm
Banner						
Harrisburg	1	4	0	0	0	
Kimball						
Bushnell	-	9	-	2	2	1,000
Dix	-	26	1	12	3	750
Kimball	2	19	-	26	-	-
Scotts Bluff						
Scottsbluff						
Gering	3	66	1	23	28	22,860
Terrytown					IND[1] 20	34,775
Henry	1	5	7	16	2	750
Lyman	5	7	12	7	3	1,030
Minatare	-	9	-	19	3	1,800
Mitchell	-	16	-	18	4	3,300
Morrill	-	21	-	48(14 IND @ 100gpm)	5	3,850
Goshen						
Fort Laramie	21	25	21	11	4	1,625[a]
La Grange	81	8	13	23	4	800[a]
Lingle	15	19	23	15	2	1,075
Torrington	158	71	165	37	12	9,900
Yoder	8	0	22	0	3	37
Platte						
Chugwater	14	1	6	-	3	180
Glendo	19	0	18	6	0	-
Guernsey	60	6	29	0	4	1,450[a]
Hartville	3	0	9	0	0	-
Wheatland	80	29	131	42	11	4,235
Laramie						
Albin	8	9	17	8	3	250
Burns	11	5	29	5	4	1,550
Pine Bluffs	21	21	24	17	7	3,775[a]

Note: a Some wells without gpm rating.

1 Industrial.

Table 2.6-39

GROUNDWATER RIGHTS WITHIN 1 MILE OF
LAUNCH FACILITY AND LAUNCH CONTROL FACILITIES

b	1 (LCF)		2		3		4		5		6		7		8		9		10		11	
	#	gpm	#	gpm	#	gpm	#	gpm	#	gpm	#	gpm	#	gpm	#	gpm	#	gpm	#	gpm	#	gpm
A	4	27	6	52.5	15	10,766	5	41[a]	2	15	2	12.5	5	37.5[a]	3	650	3	1,010[a]	3	22.5	9	929[a]
B	0	0	12	5,257	7	1,825	0	0	1	910	0	0	5	130[a]	2	7[a]	9	250	12	1,033	0	0
C	1	230	0	0	3	3,050	0	0	0	0	0	0	0	0	0	0	0	0	0	0	3	2,070
D	0	0	1	625	0	0	1	900	4	3,600	1	800	0	0	0	0	0	0	0	0	2	1,350
E	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	5	15	3	910[a]	4	47
P	0	0	3	32.5	8	30[a]	2	17	1	6	3	29.5	1	5	1	10	1	7.5	1	7.5	3	23.5
Q	6	225[a]	2	32.5	3	40	6	56.5	1	5	1	10	0	0	8	85	0	0	0	0	5	40.5
R	10	322	2	31	5	25[a]	5	30	2	17.5	3	15.5	4	63	1	17.5	4	50.5	2	10	2	27.5
S	2	20	14	157	7	470.5	17	202.5[a]	3	32	5	41	4	10[a]	2	18	9	68.5[a]	4	27	5	29
T	3	45	4	55	5	29.5	0	0	2	27.5	6	58	7	110	0	0	3	50.5	18	3,987[a]	16	3,113[a]

Note: a Indicates at least one well did not have yield information available.

b Letters designate flights, numbers refer to facilities, with 2 through 11 being Launch Facilities. The number refers to the number of water rights listings or permit listings for wells, with the gpm being the total of all such rights or listings.

Table 2.6-40

HYDROGEOLOGIC UNITS IN THE ROI

Geologic Units	Dominant Lithology	Hydrogeologic Unit	Approximate Maximum Thickness (ft)	Transmissivity (sq ft per day)	Hydrologic Role
Floodplain and terrace deposits	Unconsolidated coarse-grained sediments	Quaternary Alluvium	500	850 - 82,100	Major producing aquifer
Ogallala Formation	Poorly consolidated coarse-grained sediments	Ogallala	540	130 - 5,000	Major producing aquifer
Arikaree Formation	Fine-grained sandstone	Arikaree	1,200	8 - 10,000	Major producing aquifer
Brule Formation	Siltstone and fine-grained sandstone	White River	540	70 -105,000	Aquitard with discontinuous major producing aquifers
Chadron Formation	Siltstone and claystone	White River	700		
Lance Formation Fox Hills Sandstone	Fine-grained sandstone with some shale and coal	Lance-Fox Hills	2,050	50 - 700	Minor potential aquifer
Pierre Shale Niobrara Formation Carlisle Formation Green Horn Formation Frontier Formation Mowry Shale Muddy Sandstone Skull Creek Shale Cloverly Formation	Shale Shale Shale Limestone Shale Shale Sandstone Shale Sandstone, siltstone, and shale	Mesozoic, fine-grained sediments	10,000	--	Major aquitard with discontinuous minor potential aquifers
Morrison Formation Sundance Formation Chugwater Formation Goose Egg Formation	Shale Sandstone Siltstone Siltstone		1,900	--	Aquitard with discontinuous minor producing aquifers
Casper Formation (Hartville Formation)	Sandstone and limestone	Casper	1,200	--	Major potential aquifer
Precambrian Basement Rocks	Intrusive igneous and metamorphic rocks	Precambrian basement rocks		--	Major aquitard

Sources: Libra et al. 1981; Lowry and Crist 1967; Bjorkland 1959; Smith and Sounders 1975; DeGraw 1969.

to the Hartville Formation), Mesozoic fine-grained clastic sedimentary units, upper Cretaceous Fox Hills and Lance formations, Oligocene White River Group, Miocene Arikaree Formation, Miocene-Pliocene Ogallala Formation, and Quaternary unconsolidated alluvial deposits (Table 2.6-40). Five of these units are considered to be major aquifers within the ROI. For the purposes of this report, a major aquifer is one which is known to produce 1,000 gpm or more from a single well, while a minor aquifer is one which is known to produce more than 100 but less than 1,000 gpm from a single well. In addition, the aquifers have been designated as producing or potential, with producing aquifers yielding significant quantities of groundwater to wells within the ROI and potential aquifers being known to yield significant quantities elsewhere.

Discussions in this section focus on the following aquifers: the Quaternary Alluvium, the Ogallala, the Arikaree, the White River (consisting of the Brule and the Chadron formations), and the Lance-Fox Hills. These aquifers are presently used as sources of water within the ROI and will be the ones most heavily used in the future.

2.6.4.2 Aquifer Properties

The Fox Hills and Lance formations consist primarily of fine-grained sandstone with some interbeds and thin coal layers. The predominance of sandstone in these two units makes them a minor potential aquifer. The relatively poorly sorted nature of the majority of these sandstones reduces their hydrologic capabilities and cause these units to be less heavily utilized than the units overlying them.

The use of water from the Lance Formation is limited to domestic and stock uses because of the relatively poor yields.

Limited hydrologic data exist for the Lance-Fox Hills aquifer in the ROI. Most of the information available for this unit has been obtained in the Goshen Hole area. The development of the aquifer has occurred predominantly in this area.

Unfortunately, the direction of groundwater flow and the hydraulic gradient are unknown. It is assumed that the flow direction is to the northeast, as this unit receives recharge along the eastern flank of the Laramie Mountains and discharges into the North Platte River.

Well yields from the Lance Formation in the Goshen Hole area vary from 3.5 to 100 gpm, with an average of about 20 gpm. The average value of specific capacity is approximately 1 gallon per minute per foot (gpm/ft), with a range of 0.05 to 4.7 gpm/ft. Transmissivity and hydraulic conductivity values have been estimated from the specific capacities, ranging from 50 to 700 square feet per day (sq ft/day) and 1 to 15 feet per day (ft/day), respectively, averaging roughly 200 sq ft/day and 3 ft/day (Rapp et al. 1957).

The Oligocene White River Group is comprised of a lower and an upper unit, the Chadron and Brule formations, respectively. These two formations are composed predominantly of siltstone and fine-grained sandstone, with considerable amounts of claystone in the Chadron. As a result of its higher clay content, the Chadron Formation acts as an aquitard. Although the Brule is also rela-

tively fine-grained, it contains coarse-grained channel deposits and zones of secondary permeability in fractures that can supply large quantities of water. Consequently, this unit has been classified as an aquitard with discontinuous major producing aquifers.

The Brule Formation is present in much of the ROI, but yields large amounts of water only where fractured zones are present. Other areas yield sufficient water for stock and domestic use.

Yields and specific capacities for wells in the Brule range from yields less than 10 to over 1,000 gpm and less than 1 to over 100 gpm/ft, respectively (Libra et al. 1981). Average values for these two parameters are roughly 650 gpm and 50 gpm/ft. Estimates of hydraulic conductivity, transmissivity, and storativity have been obtained only in southern Goshen County near La Grange. Hydraulic conductivity and transmissivity values range from about 1 to 800 ft/day and 70 to 105,000 sq ft/day respectively, but the paucity of data renders any calculation of average values meaningless. Only two estimates of storativity have been made, both on the order of 0.0001 (Borchert 1976). No data are available on effective porosity.

The Arikaree Formation is present throughout most of the ROI in Wyoming, although it is missing along the western edge and along the larger river valleys, such as the Lodgepole Creek and North Platte River area. The unit is missing throughout most of the ROI in Nebraska. It is exposed to a small extent along the sides of the valleys in Banner and Scotts Bluff counties. The Arikaree Formation is most extensively used in Goshen and northern Laramie counties and Platte County below Wheatland Flats, and to a lesser extent, in southern Scotts Bluff and northern Banner counties.

The Arikaree Formation consists of fine-grained sandstone with layers of siltstone, some coarse-grained sandstone and a basal conglomerate. The formation is capable of providing sufficient water for stock, domestic, public, and irrigation uses. Those wells with larger yields generally penetrate a significant portion of the aquifer.

Well yields vary from less than 2 to over 1,000 gpm and specific capacities generally range from less than 1 to about 35 gpm/ft, although one value of 230 gpm/ft has been reported (Libra et al. 1981). Average values for well yield and specific capacity are approximately 500 gpm and 5 gpm/ft, respectively. Greater values of specific capacity (up to 30 gpm/ft) can be expected from those wells penetrating a large thickness of the aquifer.

Hydraulic conductivities range between 0.1 and 50 ft/day, but an average value would be misleading because of the lack of data. Estimates of transmissivity and storativity vary from about 8 to 10,000 sq ft/day and 0.00005 and 0.003, respectively (Libra et al. 1981). Average values for these two parameters are approximately 1,200 sq ft/day and 0.001. The above water-bearing characteristics also apply to the areas in Nebraska where the Arikaree is present.

The Ogallala Formation is lithologically similar to the Arikaree Formation, but is generally coarser and more poorly consolidated. As a result of these textural differences, the Ogallala usually yields greater quantities of water. However, the Ogallala generally is not as widely used as the Arikaree within the ROI.

The Ogallala is found in Laramie, Kimball, and Banner counties with small amounts of equivalent rocks found in Platte County. It is within the first three counties where the aquifer is most extensively used. As with the Arikaree, the Ogallala has been eroded away in the river valleys but is present at thicknesses up to 350 feet in the uplands along Lodgepole Creek.

The Ogallala is capable of delivering sufficient water for irrigation, public water supplies, and domestic use. The cities of Cheyenne, Kimball, Dix, and Bushnell obtain at least a portion of their drinking water supplies from the Ogallala.

Wells completed in the Ogallala yield from 0.6 to nearly 2,000 gpm, and average around 800 gpm. Specific capacities for these wells average about 60 gpm/ft and usually range between 0.3 and 150 gpm/ft. Both yields and specific capacities increase as one moves further east in the ACS. For example, yield and specific capacities for wells in Laramie County average 500 gpm and 10 gpm/ft, respectively. The averages for the same two parameters in Kimball County are 880 gpm and 62 gpm/ft, respectively.

Within Laramie County, hydraulic conductivity estimates average approximately 70 ft/day and generally range from about 20 to 100 ft/day (Lowry and Crist 1967), although a value of 500 ft/day has been reported (Borchert 1976). Calculated values of transmissivity and storativity in Laramie County usually vary from 130 to 5,000 sq ft/day and 0.00003 to 0.0006, respectively (Lowry and Crist 1967). However, estimates as high as 95,000 sq ft/day and 0.007 have been calculated from pump test data (Borchert 1976). Average values for these 2 parameters are approximately 2,000 sq ft/day and 0.0001.

No information regarding hydraulic conductivity and storativity within Nebraska was available. A map of transmissivity values (University of Nebraska Conservation and Survey Division 1980) indicates that transmissivity values range from 2,700 sq ft/day to 27,000 sq ft/day in the principal aquifer. In Kimball County, a typical value appears to be 6,700 sq ft/day. The principal aquifer in the area consists of the Arikaree and Ogallala groups of Tertiary age and of unconsolidated deposits of Quaternary age, so these values may not be representative of the Ogallala in Kimball County.

The Quaternary alluvial deposits primarily consist of unconsolidated coarse-grained materials deposited by existing and prehistoric streams, although some silt and clay layers are also present. The lithologic nature of these deposits makes them important aquifers where they are saturated. However, the extremely limited areal extent and saturated thickness greatly decrease their regional usefulness as a major groundwater source.

The Quaternary Terrace and alluvial deposits have been reported along 1) the floodplains of the major drainages in the ROI, specifically Pumpkin, Horse, Lodgepole, and Crow creeks and the North Platte and Laramie rivers; 2) the Wheatland Flats area; and 3) the prehistoric terraces in the southeastern portion of Laramie County. These deposits can reach maximum thickness of up to 200 feet in Goshen and Scotts Bluff counties but usually average about 25 to 50 feet.

Yields and specific capacities for wells completed in the Quaternary deposits generally vary from about 30 to over 3,400 gpm and 1 to 250 gpm/ft,

respectively, and average around 500 gpm and 45 gpm/ft. Hydraulic conductivity values are consistently higher than they are for the other hydrogeologic units, ranging from 70 to 1,300 ft/day, with an average of approximately 330 ft/day. Also, within Wyoming transmissivity and storativity estimates are significantly greater than for the other units, with ranges of 850 and 82,100 sq ft/day and 0.0045 to 0.235, respectively (Libra et al. 1981), although one well reportedly test pumped for 37 hours at a discharge of 3,400 gpm had a calculated transmissivity of 221,000 sq ft/day (Rapp et al. 1957). Average values of transmissivity and storativity are about 13,000 sq ft/day and 0.05, respectively.

Data on hydraulic conductivity, transmissivity, and storativity are only available for the Wyoming counties within the ROI with a few exceptions. Values of transmissivity taken from the map mentioned earlier range from 2,700 to 27,000 sq ft/day in the principal aquifer. Typical values range from 6,700 to 13,400 sq ft/day. Also, a hydraulic conductivity value of 190 ft/day was reported in Scotts Bluff County (Wenzel et al. 1946). These values are all within the range of values found in Wyoming and therefore the ranges and averages given for Wyoming will be considered valid for the ROI in Nebraska. As in the Ogallala aquifer, the values for the hydraulic properties tend to increase as one moves east within the ROI. This phenomenon probably reflects the increase in the saturated thickness of the Quaternary deposits from west to east.

The Quaternary deposits are used as a source of water for irrigation, municipal, and domestic purposes in those areas where the aquifer has a large saturated thickness.

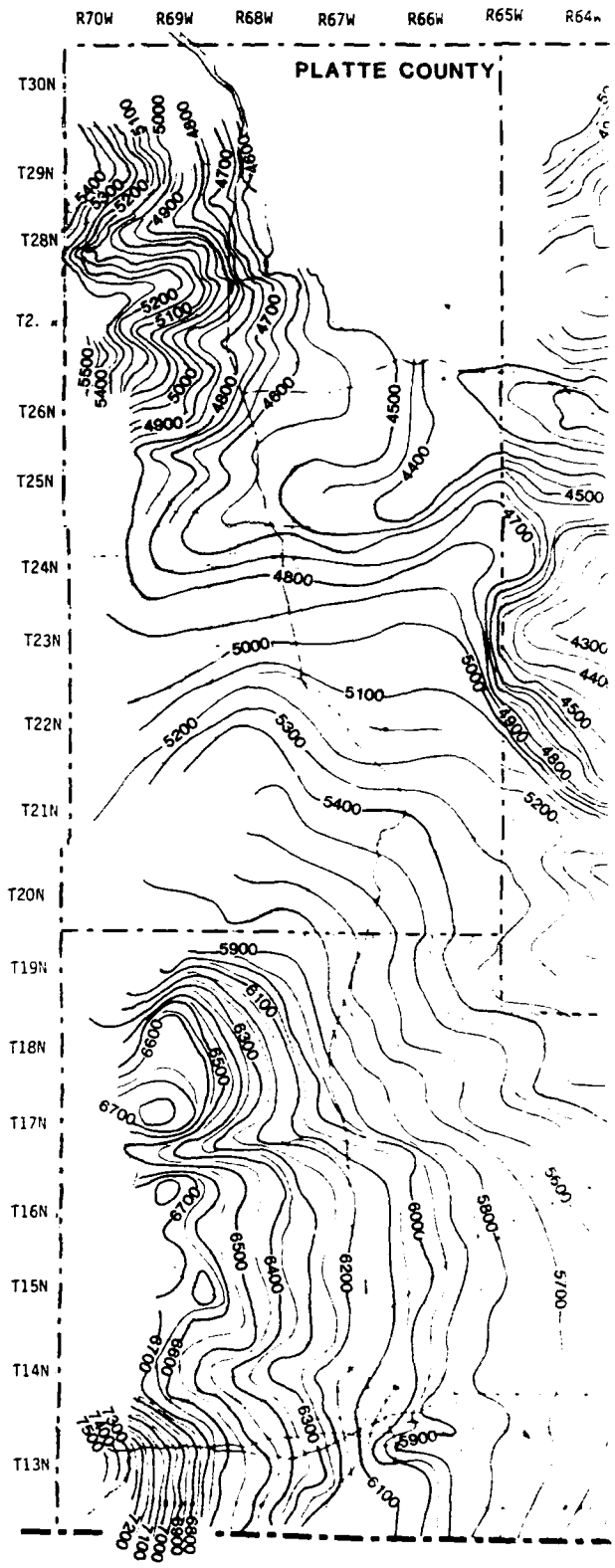
2.6.4.3 Groundwater Hydrology

2.6.4.3.1 Regional Groundwater Flow

Figure 2.6-17 shows an approximation of the groundwater table elevations throughout the ROI. Because the groundwater table is constantly changing, both seasonally and long term, it is impossible to give a precise illustration of the groundwater table.

As is seen in the figure, the groundwater table elevation varies from about 7,500 feet in the southwest corner of Laramie County, to approximately 3,800 feet near the North Platte River in eastern Scotts Bluff County. Groundwater flow is perpendicular to the contours of the water table. Velocity is inversely proportional to the distance between the contours, meaning that water velocity increases as distance between the contours decreases. Velocity is dependent on the hydraulic conductivity of the aquifer.

Direction of groundwater flow is generally to the east and as one moves north out of the Lodgepole Creek Basin, the water also flows towards the North Platte River. In this basin, much of the flow reaches the North Platte River either directly from the groundwater or by first entering a tributary and then flowing to the North Platte.



SCALE IN MILES

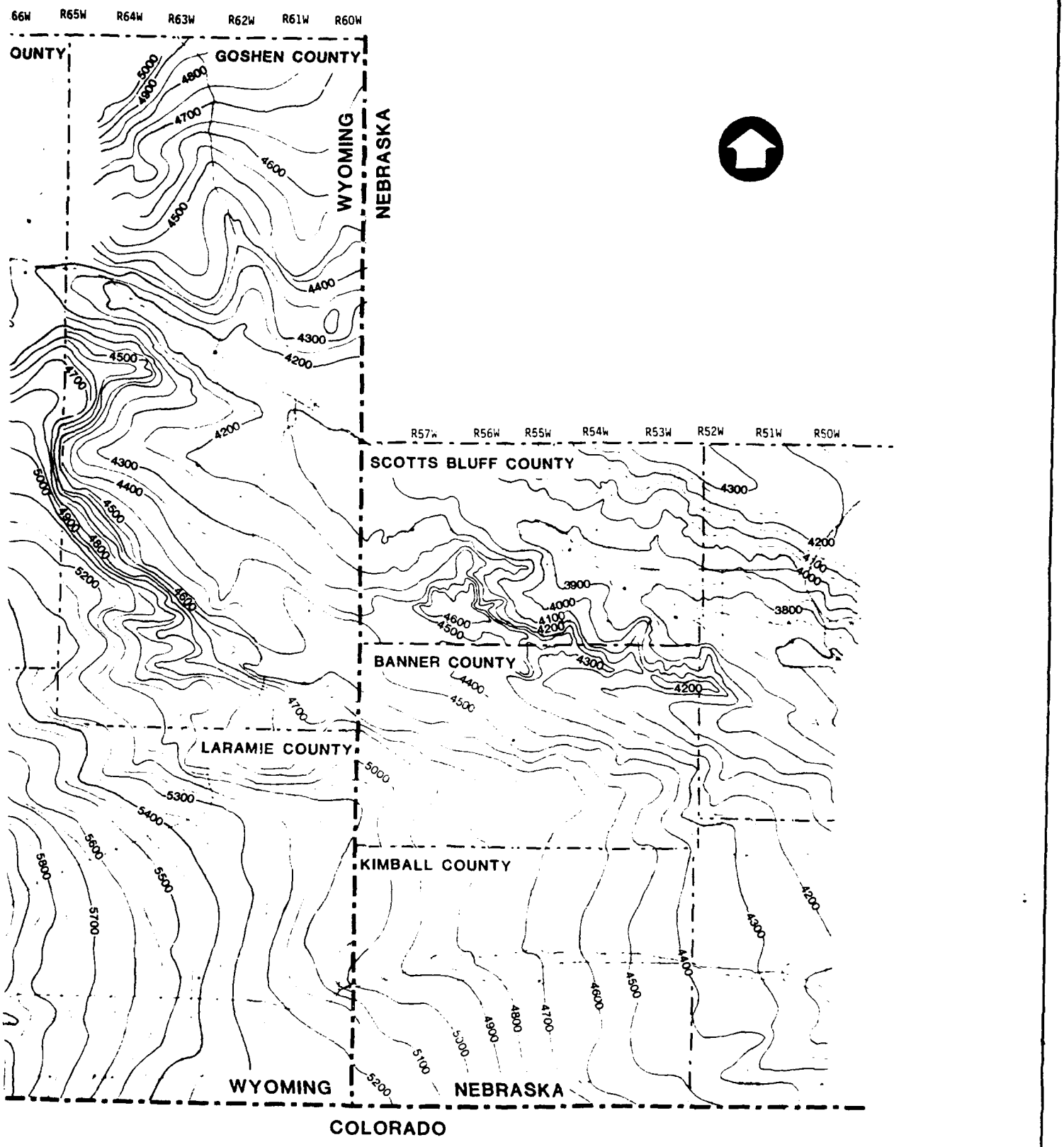
0 5 10 20

LEGEND

—4300— CONTOUR ELEVATION (FT)

APPROXIMATE GROUNDWATER TABLE CONFIGURATION WITHI

(1)



N WITHIN THE ROI

FIGURE NO. 2.6-17

(2)

2.6.4.3.2 Crow Creek Watershed Area

The principal aquifers which supply water within the Crow Creek watershed area are the Brule (White River Formation), the Arikaree, the Ogallala, and Quaternary deposits. Typical hydraulic properties, water quality, and distribution of each of the aquifers will be discussed below.

The Brule Formation reaches a thickness of 400 feet in the Crow Creek watershed. It supplies water for irrigation, municipal, industrial, domestic, and stock uses in the area. Large yield wells are found in the fractured zones of the aquifer. Very little information on the water-bearing characteristics is available for this area. One well in the CBPU's Federal well-field will discharge 450 gpm. No basin-specific information is available for the other parameters.

The Arikaree Formation has been removed by erosion throughout much of the watershed. It is exposed along the edge of the uplands of the Crow Creek Valley and on the south-facing escarpment west of Carpenter. Maximum thickness is approximately 500 feet. Because of low permeability and a thin zone of saturation, large yield wells for irrigation are not possible but the formation does yield sufficient water for domestic and stock uses. Basin-specific information for water-bearing properties of the Arikaree is not available. Two wells that are in contact with both the Arikaree and Ogallala yield water in excess of 300 gpm (Lowry and Crist 1967).

The Ogallala is the most extensively developed aquifer in the Crow Creek watershed. It reaches a maximum thickness of greater than 320 feet and is the surficial formation throughout much of the basin. The Ogallala provides water pumped from the Cheyenne wellfields. Discharge for wells tapping the Ogallala yield between less than 1 to almost 500 gpm, with an average of about 330 gpm. Specific capacity ranges between 2 to over 16 gpm/ft. Average specific capacity is approximately 6.6 gpm/ft. Transmissivity in the basin averages about 2,100 sq ft/day with a range of 150 to 4,600 sq ft/day. A typical value for hydraulic conductivity is approximately 65 ft/day, and it ranges between 22 and 100. Storativity is about 2.5×10^{-4}, with a range of 5.9×10^{-5} to 5.6×10^{-4}. Yield and specific capacity for the Ogallala in this area are somewhat lower than the averages for the entire ROI. This is probably related to the thickness of the zone of saturation. The remaining parameters all have about the same range and averages as seen throughout the remainder of the ROI.

Large quantities of water are pumped from wells tapping Quaternary deposits in the Carpenter area. The aquifer reaches maximum thickness of 30 to 50 feet. Average discharge from the Quaternary deposits is approximately 970 gpm and ranges from 40 to 2,000 gpm. Specific capacity varies between 8 and 72 gpm/ft and averages 36 gpm/ft. Averages for the remaining parameters were not calculated due to lack of data but ranges are: transmissivity, 5,900 to 20,000 sq ft/day; hydraulic conductivity, 90 to 425 ft/day; and storativity, 4.65×10^{-3} to 5.43×10^{-2}. The upper end values of these ranges are less than the upper end of the ranges for the entire ROI, but the values appear to be typical.

While the Casper Formation is not currently used as a source of groundwater, it does contribute to the flow of Lodgepole and Crow creeks via springs west

and southwest of Cheyenne. One of the springs yields almost 2 cfs (900 gpm). Most of this flow is used for irrigation downstream in the Crow Creek watershed.

A groundwater budget and recharge and discharge estimates for the post-Cretaceous aquifers in the Crow Creek watershed are presented in Table 2.6-41. From these computations it is apparent that the post-Cretaceous aquifers receive recharge primarily from precipitation, but also gain a significant amount of water from streams originating in the Laramie Mountains to the west and from underflow. It is estimated that approximately 0.83 inches or about 5 to 6 percent of the annual precipitation reaches the groundwater system as recharge (Rapp et al. 1953). Crow Creek is generally a losing stream in the area, as evidenced by its change from a perennial to an intermittent stream (Crist 1980).

Discharge from the post-Cretaceous aquifers primarily occurs through well pumpage, springs and seeps, and underflow to other regions, as well as leakage to streams. The relative importance of each of these types of discharge is not well known in the watershed, but pumpage and underflow appear to account for the vast majority of water discharged from the post-Cretaceous aquifers.

2.6.4.3.3 Lodgepole Creek Watershed Area

As in the Crow Creek watershed, the Brule, the Ogallala, and Quaternary aquifers are the major aquifers in the Lodgepole Creek Basin within the ROI. The Arikaree is present to some extent in the upper Lodgepole Creek valley in Wyoming but has been eroded away in the lower valley in Nebraska.

The Brule Formation may reach a thickness of up to 330 feet in the area. Well yields from the Brule range from 200 to 2,000 gpm and average about 780 gpm. Such yields can provide water for irrigation, although the Formation is also used for municipal and industrial wells also. Average specific capacity is approximately 45 gpm/ft, and ranges from less than 10 to 257 gpm/ft. No other parameter values are available for the Brule in the Lodgepole Creek Basin.

Because there are very few known wells which tap the Arikaree in the basin, it is not discussed in this section.

The Ogallala is the most developed aquifer in the Lodgepole Creek watershed. It reaches thicknesses of 350 feet. Average discharge is 890 gpm and varies from 15 to 1,800 gpm. Specific capacity ranges between 1 and 400 gpm/ft, and averages 65 gpm/ft. Averages will not be calculated for the other parameters but ranges are: transmissivity, 230 to 94,600 sq ft/day; storativity, 3.2×10^{-5} to 6.7×10^{-5}; and a single hydraulic conductivity value of 535 ft/day.

Quaternary deposits are important aquifers in the Lodgepole Creek watershed in the southeast portion of the basin in Wyoming (the Pine Bluffs lowlands) and in the floodplain and under the two terraces of the valley in Nebraska. The deposits are typically 25 feet thick in Nebraska. Well yields range from 400 to 1,500 gpm, providing sufficient water for irrigation. Average yield is about 800 gpm. Yields of this magnitude are only found where the water table is near the ground surface, thereby providing a relatively large saturated thickness. Average specific yield is approximately 50 gpm/ft, ranging from

Table 2.6-41

ESTIMATED GROUNDWATER BUDGETS FOR CROW CREEK WATERSHED AREA IN 1980

	acre-ft/yr
Recharge	
Precipitation[1]	16,600
Underflow[2]	500
Streamflow[3]	4,000
TOTAL:	21,100
Discharge	
Irrigation From Groundwater[4]	9,400
Municipal and Industrial Water Use[5]	3,800
Phreatophytes[6]	1,400
Evaporation[7]	1,000
Groundwater Outflow[8]	4,000
Surface Water Outflow	0
TOTAL:	19,600
Change in Groundwater Storage[9]	+1,500

Notes:
1. Based on 5% of 15" precipitation over basin area.
2. Estimated flow from Lone Tree Basin to Crow Creek.
3. Based on seepage runs made in 1964 adjusted for increased wastewater discharges downstream of Cheyenne which have increased this value since then for Crow Creek.
4. Based on distribution of irrigated lands in Laramie County that were irrigated in 1980 with groundwater.
5. Based on groundwater use records for 1980.
6. Based on estimates in Main Report Platte River Basin, distributed to watersheds based on riparian vegetation existing in 1980 (USDA 1980).

Table 2.6-41 Continued, page 2 of 2
ESTIMATED GROUNDWATER BUDGETS FOR CROW CREEK
WATERSHED AREA IN 1980

7 Based on estimates from Main Report Platte River Basin (USDA 1980).

8 Outflow to Colorado in Carpenter area for Crow Creek.

9 Difference between recharge and discharge. Total groundwater in storage in Laramie County is about 50 million acre-ft (Lowry and Crist 1967). Seasonal groundwater declines of as much as 40 feet occur around the Cheyenne wellfields, with lesser seasonal declines in eastern Laramie County.

11 to 200 gpm/ft. Values for the remaining parameters are not available for this basin.

2.6.4.3.4 Horse Creek Watershed Area

Aquifers of importance in the Horse Creek watershed area are the Lance, the Brule, the Arikaree, the Ogallala, and Quaternary deposits. The Chadron aquifer yields water in some portions of the basin but only in small amounts for stock and domestic uses.

The Lance aquifer yields only small amounts of water in Laramie County but delivers up to 100 gpm in the basin in Goshen County. Both municipal and industrial wells tap the Lance at Yoder, Wyoming. Most of the wells obtaining water from the Lance are in south-central Goshen County (the Goshen Hole lowlands area). Average yields from wells in that area are 20 gpm, with a minimum of about 4 and a maximum of 100. Specific capacity for the same wells averages 1.2 gpm/ft, ranging from 0.05 to 4.7 gpm/ft. Transmissivity and hydraulic conductivity ranges are 4.5 sq ft/day to 270 sq ft/day and 1 to 17 ft/day, respectively. The transmissivity values have been estimated from specific capacity values (Libra et al. 1981).

The Brule Formation reaches a thickness of 450 feet in this watershed. The aquifer is found throughout Laramie County and in southern and central Goshen County. Large yields are obtained from the fractured zones in the aquifer. Discharge will range from less than 5 to over 1,000 gpm, and averages 550 gpm. Specific capacity varies between less than 1 to 130 gpm/ft, with an average of 3.5 gpm/ft. Transmissivity ranges from 65 to 100,000 sq ft/day, but there are too few values to obtain a meaningful average. Storativity is on the order of 1×10^{-4}, and hydraulic conductivity ranges from 700 to 800 ft/day.

The Ogallala is not found in most of the Horse Creek watershed. It is present in the western portion of Laramie County but is not greatly developed. Very little information on wells tapping the Ogallala in the basin is available and therefore the Formation will not be discussed further in this section.

The Arikaree is the major aquifer in the Horse Creek watershed area. It is found in the uplands area of the basin, such as in the southwest corner of Goshen County and north and northeast Laramie County. It reaches a thickness of 500 feet in these areas. Unfortunately, very little information exists on the Arikaree in this area. Average discharge is approximately 250 gpm, and can vary between 1 and 1,000 gpm. Specific capacity ranges from less than 1 to 23 gpm/ft and averages about 7 gpm/ft. An average for transmissivity is not available due to lack of data but ranges from 1,200 sq ft/day to 2,900 sq ft/day.

The Quaternary deposits are not present to a great extent in the Horse Creek watershed. Terrace deposits are found along the eastern edge of the basin but this formation is not very large and little is known about it. Well yields in that area average 460 gpm and range from 30 to 840 gpm. Specific capacities vary between about 1 to 60 gpm/ft and average 26 gpm/ft. Limited data are available for transmissivity and storativity but they range from 17,000 to 50,000 sq ft/day and 5×10^{-3} to 2×10^{-2}, respectively.

2.6.4.3.5 Pumpkin Creek Watershed Area

The major aquifers in the Pumpkin Creek watershed area are the Chadron, the Brule, the Pierre, the Arikaree, the Ogallala, and Quaternary deposits with most wells developed in the Brule or in the alluvium. Very little information about each aquifer is available in the watershed other than thickness and some values of well yield. This information is presented in tabular form below.

	Maximum Thickness (ft)	Discharge (gpm)
Chadron	170	Up to 1,000[a]
Brule	540	Up to 1,000[b]
Arikaree	460	500
Ogallala	540	N/A
Quaternary	105	500
Pierre	N/A	N/A

NOTE: a Such a large discharge would be accompanied by drawdowns of 200 to 500 feet (Smith and Souders 1975).
b Brule yields require locations where formation is fractured. Wells may have large yields in spring but only small yields in late summer due to limits in formation storage (Gottula, 1980).
N/A Data not available

2.6.4.3.6 Chugwater Creek Watershed Area

Only three aquifers are major water suppliers in the Chugwater Creek watershed area. These are the Brule, the Arikaree, and the Quaternary deposits. Much of the information on the Arikaree in this basin comes from the Wheatland Flats area.

The Brule yields only small quantities of water for stock and domestic use. Only one value for discharge is available, one well discharging 3.9 gpm with a specific capacity of 0.8 gpm/ft.

The Arikaree is present throughout much of the area, with a maximum thickness of about 1,100 feet. Sufficient water for irrigation is provided by some wells in the area. Discharge ranges from 150 to 700 gpm for irrigation wells. Other wells provide enough water for domestic, stock, or municipal use.

The Quaternary deposits are probably the most used of the aquifers in the basin. They reach thicknesses of 85 feet in the Wheatland Flats area (Morris and Babcock 1960). Yields vary between 40 and 700 gpm. There are an insufficient number of values to calculate an average yield. Specific capacity ranges from 2.5 to 12 gpm/ft. Information on the other parameters is not available.

2.6.4.4 Groundwater Quality

The availability, reliability, and spatial distribution of water quality data varies greatly for aquifers in the ACS. As with the hydrologic properties, information on groundwater quality decreases with increasing in depth from the

from wells completed in the post-Cretaceous aquifers. Table 2.6-42 summarizes the water quality data available for the aquifers in the ROI.

Water quality information for the Casper Formation is sparse and limited to outcrop areas along the eastern flank of the Laramie Mountains. Groundwater obtained in those areas generally contains less than 500 mg/l TDS, predominantly calcium and bicarbonate (Western Water Consultants 1982). Water quality data collected from the Hartville Formation (Casper equivalent) in Platte County indicate that TDS content increases away from outcrop, and the groundwater changes to a calcium sulfate and then sodium sulfate composition. Constituents which are known to exceed primary or secondary drinking water standards in areas of high salinity are sodium, sulfate, chloride, and fluoride (Western Water Consultants 1982).

Much of the water quality information for the Lance/Fox Hills aquifer was gathered from the Goshen Hole area in Platte County. TDS concentrations of samples taken in the Goshen Hole area range from about 400 to 4,000 mg/l. Major ion composition of groundwater in the Lance-Fox Hills hydrogeologic unit changes from predominantly calcium bicarbonate to sodium bicarbonate with increasing TDS concentrations. Chemical composition of one sample taken from a spring in the Goshen Hole area suggests that sulfate may replace bicarbonate as the dominant anion in water with a TDS content of more than 1,000 mg/l, although such a generalization is tentative given the limited data (Rapp et al. 1957).

In the Goshen Hole area fluoride and sulfate have been found to exceed primary or secondary drinking water standards in groundwater samples obtained from the Lance aquifer, with maximum reported values of 3.4 and 405 mg/l, respectively. Radium-226 concentrations in the few samples that have been analyzed for radioactive species are all well below the primary drinking water standard, reaching a maximum reported value of only 0.28 picoCuries per liter (pCi/l); however, uranium levels are elevated above values normally found in natural waters with reported levels as high as 0.27 mg/l (Libra et al. 1981).

TDS concentrations in the Brule Formation range from 168 to over 1,000 mg/l. Major ion composition is calcium bicarbonate in low TDS waters and sodium bicarbonate in high TDS waters.

Two groundwater samples from the White River Group in the Goshen Hole area exceed the primary drinking water standards for nitrate, with a maximum reported value of 94 mg/l (Rapp et al. 1957). No other samples have been found to exceed any primary standards. However, a number of samples in the western and southeastern portions of Laramie County exceed the secondary drinking water standard for iron, reaching a maximum reported value of 1.1 mg/l (Rapp et al. 1953, Bjorklund 1959). Excessive levels of chloride and sulfate, as well as iron, have been reported for groundwater in the vicinity of Goshen Hole, which reach maximum levels of 268 and 486 mg/l, respectively (Rapp et al. 1957).

Limited data on radioactive species have been collected for groundwater from the White River Group in the Goshen Hole area. Concentrations of radium-226 vary from 0.10 to 0.24 pCi/l, well below the primary drinking water standard of 5 pCi/l. However, relatively high levels of uranium are present, ranging from about 0.04 to 0.20 mg/l (Libra et al. 1981).

Table 2.6-42

WATER QUALITY OF AQUIFERS

Parameter	Units	Quaternary Alluvium			Arikaree[1]			Ogallala[1]			White River (Brule)[1]			Lance-Fox Hills			Casper		
		Min.	Avg.[2]	Max.	Min.	Avg.[2]	Max.	Min.	Avg.[2]	Max.	Min.	Avg.[2]	Max.	Min.	Avg.[3]	Max.	Min.	Avg.[3]	Max.
Chloride (Cl)	mg/l	1.0	19	129	2.0	10	22	1.0	7.0	38	2.0	13	114	5.2	N/A	169	1.8	6.0	29
Fluoride (F)	mg/l	0.2	0.55	1.3	0.2	0.62	1.4	0.0	0.49	0.9	0.2	0.66	1.6	0.3	1.48	2.8	0.2	0.65	1.2
Hardness (as $CaCo_3$)	mg/l	80	316	872	69	191	362	47	153	230	7.0	151	332	5.0	47	171	90	170	249
Nitrate (asN)	mg/l	0.04	3.8	141	0.04	2.6	6.8	0.0	1.8	9.0	0.0	2.1	22	0.0	1.3	8.8	0.02	0.4	1.1
Sodium (Na)	mg/l	10	80	616	6.4	21	67	4.4	N/A	42	8.2	80	324	13	N/A	429	18	24	45
Sulfate (SO_4)	mg/l	0.3	211	975	0.0	31	172	0.3	12	53	3.8	38	298	0.9	70	405	1.6	40	146
TDS	mg/l	200	566	1,410	22	523	664	70	279	2,012	168	445	1,460	231	910	4,076	60	336	2,897
Major ion composition																			
Low TDS	-	Calcium-Bicarbonate			Calcium-Bicarbonate			Calcium-Bicarbonate			Calcium-Bicarbonate			Calcium-Bicarbonate			Calcium-Bicarbonate		
High TDS	-	Sodium-Sulfate			Calcium-Bicarbonate			Calcium-Bicarbonate			Sodium-Bicarbonate			Sodium-Bicarbonate			Calcium-Bicarbonate		
Radium-226	pCi/l		N/A			N/A			N/A		0.1	N/A	2.4	0.01	N/A	0.28		N/A	
Uranium	mg/l		N/A			N/A			N/A		0.038	N/A	0.02	0.015	N/A	0.27		N/A	

Notes: 1 Some samples from these aquifers also exhibit iron concentrations exceeding the secondary drinking water standard.

2 Average values for samples from Nebraska - N/A indicates minimum and maximum values occur in separate states.

3 Average values for samples from Wyoming - N/A indicates minimum and maximum values occur in separate states.

Source: Ertec Western 1983a, 1983b.

These concentrations are higher than the typical range for natural water, although no drinking water standards have been established for uranium.

No major differences in groundwater quality are apparent between the Arikaree and Ogallala aquifers; therefore, they will be described together. More data exist for these two hydrogeologic units than any of the others.

Water quality in the Arikaree and Ogallala formations is typically good. TDS concentrations throughout the ROI are generally below 500 mg/l. No obvious relationship exists between TDS levels and sample location or depth. It seems likely that local differences in TDS are related to textural changes in the relatively coarse-grained clastic sediments comprising these two lithologic units. This means that low TDS concentrations probably are associated with coarse-grained channel deposits, and the finer-grained sandstones comprising the majority of these units are likely sources of higher TDS groundwater.

Calcium and bicarbonate are the major ions found in groundwater samples taken from the Arikaree and Ogallala formations (Babcock and Bjorklund 1956, Lowry and Crist 1967). Unlike most of the other aquifers in the ROI, there is no apparent regional trend in major ion composition for groundwater in these two lithologic units. This may be the result of the high permeability of these units and the consequent rapid circulation of groundwater. Such an environment would result in relatively good quality water with a reasonably uniform chemical composition.

Groundwater samples from the Arikaree and Ogallala formations in the Cheyenne vicinity frequently exceed the primary drinking water standard for nitrate (Babcock and Bjorklund 1956, Lowry and Crist 1967). Concentrations as high as 150 mg/l have been reported, as compared to the maximum contaminant level of 10 mg/l. Many of the highest nitrate levels have been attributed to substandard lot size, insufficient well-septic tank spacing and the presence of livestock (Libra et al. 1981). However, of greater regional importance are the excessive nitrate levels found in areas away from Cheyenne, as extensive farming and irrigation are conducted in those areas. It is suspected that the use of fertilizers is responsible for the excessive nitrate levels (Lowry and Crist 1967).

In 1980, about 8,000 acre-ft/year of wastewater was discharged from Cheyenne wastewater treatment facilities of which about 4,000 acre-ft provided recharge to the groundwater basin. Groundwater quality data suggest that existing nitrogen concentrations upstream of Cheyenne are 0.1 to 1 mg/l, whereas they are 2 to 3.8 mg/l in wells near Crow Creek downstream of Cheyenne (STORET 1983). The TDS levels are about 280 mg/l and 300 mg/l for these areas, respectively. Crow Creek surface water quality data indicates average concentrations of 20 mg/l of nitrogen and about 400 mg/l of TDS downstream of Cheyenne. Total recharge in the watershed is estimated at 21,000 acre-ft/yr. Assuming that the 4,000 acre-ft of wastewater effluent recharges with concentration of 20 mg/l TN and 400 mg/l TDS and that the remaining 17,000 acre ft/yr recharge at 0.1 mg/l TN and 280 mg/L TDs, the resulting mass balance would indicate resultant concentrations of 3.9 mg/l TN and 303 mg/l TDS in groundwater downstream of Cheyenne. This measure of change greatly simplifies the natural system and solute transport in groundwater, but appears to approximate available water quality data.

No other chemical constituents are known to exceed primary or secondary drinking water standards within the county, with the exception of some groundwater samples from wells completed into the Ogallala Formation which contain excessively high iron concentrations. A maximum value of 5.8 mg/l has been reported (Libra et al. 1981). Radioactive analyses have not been performed on Arikaree or Ogallala groundwater samples in the ROI.

Groundwater quality in the Quaternary alluvial deposits generally is good, with TDS concentrations ranging between 200 and 1,400 mg/l (Libra et al. 1981). The primary dissolved constituents are calcium and bicarbonate in low TDS water, and in higher TDS waters sodium and sulfate are also important constituents. High nitrate concentrations in the Goshen Hole area exceed primary drinking water standards. Some tests on the radioactivity of the groundwater were conducted but the results seem inconclusive (Libra et al. 1981).

In Laramie County and southern Goshen County, Quaternary aquifer composition is similar to that of water in the underlying Tertiary aquifers. This is due to either good communication between the aquifers or lithologic similarity of Quaternary and Tertiary deposits. In central Platte and Goshen counties there is little similarity between Quaternary and Tertiary aquifer system waters. In central Platte County, water movement is downward from Quaternary aquifers to the Arikaree (Weeks 1965), whereas in Goshen County, discharge from bedrock aquifers to the alluvial aquifer is considered insignificant compared to other recharge mechanisms (Crist 1979).

The following sections point out any significant deviations from the general groundwater quality descriptions in each watershed area. Only the Horse and Lodgepole Creek watersheds show any significant deviations.

2.6.4.4.1 Lodgepole Creek Watershed Area

Samples taken from the Fox Hills/Lance aquifer show much lower TDS concentrations in this area than those found in the Goshen Hole area in Goshen County (Libra et al. 1981). The reason for this may be that TDS concentrations increase toward probable discharge areas in Goshen Hole, although potentiometric data are insufficient to substantiate this hypothesis.

A significant increase in TDS content has been reported for one well tapping Quaternary deposits in the southeastern portion of Laramie County (Lowry and Crist 1967). The increase in TDS is considered to be a result of irrigation, but this trend cannot be confirmed because few wells have had multiple analyses.

2.6.4.4.2 Horse Creek Watershed Area

TDS concentrations of samples taken from the Brule Formation are generally less than 300 and 400 mg/l in the western and southeastern portions of Laramie County, respectively (Rapp et al. 1953, Bjorklund 1959), but concentrations in excess of 1,000 mg/l have been found in the vicinity of Goshen Hole to the northeast. The hypothesis that TDS content increases towards a regional discharge zone in the Goshen Hole area is supported by the shape of the composite potentiometric surface for the post-Cretaceous aquifers, which indicates flow to the northeast (Figure 2.6-16).

Groundwater samples obtained from the Brule Formation in the western and southeastern portions of Laramie County are calcium bicarbonate to sodium bicarbonate in type (Rapp et al. 1953, Bjorklund 1959), while samples taken in the northeastern portion are predominantly of sodium bicarbonate type (Rapp et al. 1957). Limited data from the Goshen Hole area suggest that sulfate may replace bicarbonate as the major anion in samples with relatively high TDS concentrations.

3.0

ENVIRONMENTAL CONSEQUENCES, MITIGATION MEASURES, AND UNAVOIDABLE IMPACTS

3.0 ENVIRONMENTAL CONSEQUENCES, MITIGATION MEASURES, AND UNAVOIDABLE IMPACTS

This section initially presents analytic methods used to calculate future conditions both with and without the proposed project. Assumptions and assumed mitigation methods are then presented. Criteria to determine the level and significance of project effects are then defined. Analytic methods are then applied to define the environmental consequences of the No Action Alternative and project alternatives. Finally, mitigation measures are developed for significant project impacts and unavoidable adverse impacts are presented.

Figure 3.0-1 shows the location of F.E. Warren AFB and the existing Minuteman silos that would be affected by the Proposed Action. Most of the environmental consequences of the Proposed Action would occur during the construction period of 1984 to 1989, but the impacts are also considered for the operational period that could extend beyond the year 2000.

3.1 Analytic Methods

3.1.1 Water Use and Demand

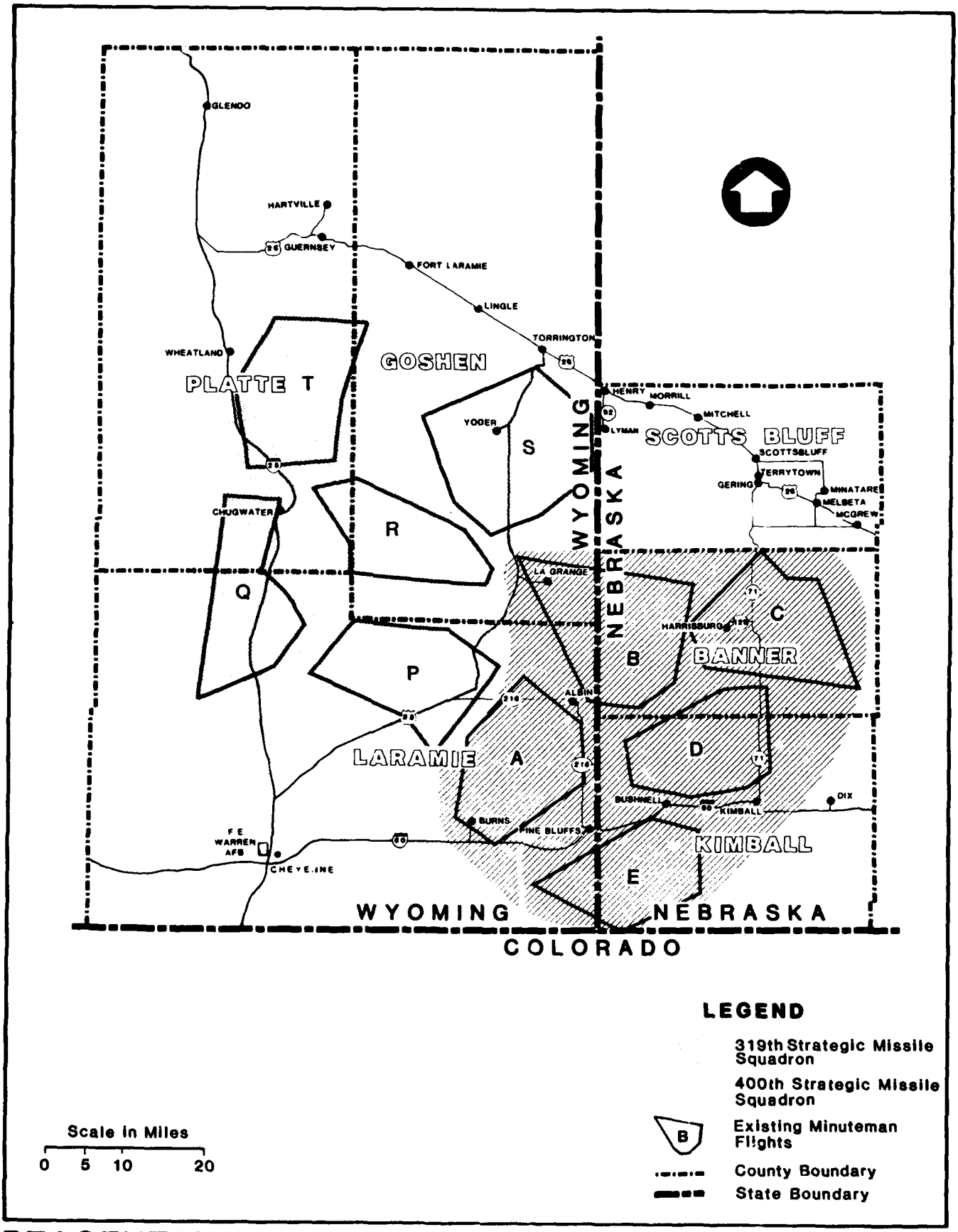
The Areas of Concentrated Study (ACS) for water use and demand include the Crow Creek watershed and those portions of counties in the Deployment Area (DA) (Banner, Kimball, Scotts Bluff, Laramie, Platte, and Goshen) in the overall Region of Influence (ROI). No direct project water demands are expected outside of the defined area.

Municipal water supplies in Albin, Burns, Bushnell, Cheyenne, Chugwater, Fort Laramie, Gering, Kimball, LaGrange, Lyman, Pine Bluffs, Scottsbluff, Torrington, Wheatland, and Yoder, as well as a commercial water supply near Harrisburg (Sec. 20, T19N, R55W), were recommended to supply construction water during initial Minuteman construction (Porter and O'Brien 1962) with water trucked to the sites. Based on initial estimates of construction needs and induced water demand, the increase in water use and demand is expected to be large only in the Crow Creek watershed ACS. Effects of meeting water demands could also affect other water resource elements in that ACS. The results of meeting water demands in the other ACSs will likely be quite localized, particularly upon municipal water supply.

3.1.1.1 Baseline Future - No Action Alternative

Future water use and demand without the project were forecast to provide a baseline for impact assessment of project-related water demands. Project water demands were superimposed on these demands to evaluate effects.

For the Crow Creek watershed, municipal water demand was based on past trends in the Cheyenne Urban Area. Past trends indicate a total water use of about 220 gallons per capita per day (gpcd) of which 180 gpcd is for nonindustrial municipal use. Industrial water use is not expected to increase at the same rate as population. A nominal increase of 100 acre-feet per year (acre-ft/yr) for industrial use plus 180 gpcd for population changes is used to forecast water demand.



PEACEKEEPER DEPLOYMENT AREA FIGURE NO. 3.0-1

Past conditions have been assumed to continue in the future for nonmunicipal, commercial, industrial, and livestock uses. Residential nonmunicipal use would be 100 gpcd. Agricultural irrigation use is estimated as 1980 use plus either 1.5 acre-feet per acre per year (acre-ft/acre/yr) or existing use rates for new lands to be irrigated.

For the other ACSs, existing conditions have been assumed to continue in the future except for municipal and agricultural irrigation demands, which are based on population changes multiplied by existing typical use in the ROI (a value of 250 gpcd) and estimates of future water demands for agricultural irrigation derived from secondary sources.

3.1.1.2 Proposed Action

Direct water requirements for project construction and operation were developed by the Air Force Regional Civil Engineer (AFRCE), the Corps of Engineers (COE), and others, based on knowledge of physical facilities to be constructed and operated. These direct project requirements have been incorporated into water development plans and programs for the project.

Indirect or induced water demands are developed based on population and land use changes that would occur in the ACSs with the project. The indirect or induced water demand was based on population and land use unit factors developed to assess future trends without the project. These unit water demand factors were applied to projected population and land use changes with the project in order to quantify water demands and use. In addition, a rate of 100 gpcd was used to estimate water use of weekly commuters and transients.

3.1.2 Constraints on Water Use

The ACS for constraints on water use includes Wyoming and Nebraska requirements and constraints relative to water use and water quality. The Air Force and/or its contractors will follow state water law in obtaining water for project construction. Project construction will only occur in portions of Nebraska and Wyoming. Water quality controls relative to construction also apply in those states. Changes in present hydrologic conditions due to induced water demands are only expected to be significant in the Crow Creek watershed.

3.1.2.1 Baseline Future - No Action Alternative

The methodology used to determine future trends is based on discussions with state officials to determine expected changes in legal requirements.

3.1.2.2 Proposed Action

Analysis of impacts delineates existing water rights that would be affected by the proposed project (i.e., Wyoming guidelines require that existing groundwater rights be inventoried within 3 miles of a proposed project area boundary or the area of site influence, whichever is greater). Proposed project facilities are evaluated to ensure compliance with legal requirements.

In addition to the above criteria, various state agencies have developed guidelines delineating preferred groundwater and surface water evaluation

techniques. For example, the Wyoming Department of Environmental Quality (WDEQ) has developed "Sedimentation Pond Review Criteria" which include specific methods to determine soil erosion and stormwater runoff for a project. Such guidelines are summarized and applied to quantify changes from baseline conditions for hydrologic-related project activities.

3.1.3 Surface Water Hydrology and Quality

The ACS for surface water hydrology includes the Crow Creek watershed upstream of the Carpenter area and the Lodgepole, Horse, Pumpkin, and Chugwater Creek watersheds in the DA.

Water quality and quantity effects on the surface water system are largely restricted to the Crow Creek watershed where population and land use changes may be sufficient to cause stormwater-related effects and wastewater discharge effects. Project-related construction effects are not expected to measurably alter streamflow patterns in the rest of the ROI. The localized changes in surface water hydrology or quality anticipated outside the Crow Creek watershed would be mainly construction-related surface water effects, especially erosion and sedimentation. Increases in wastewater discharges due to population and land use variations are not expected to cause large effects in the other ACSs or the ROI.

3.1.3.1 Baseline Future - No Action Alternative

Surface water hydrology and quality are determined by many factors. It is assumed that the factors determining existing hydrology and quality will continue in the future. Where human activity can be identified that would alter those factors, a qualitative analysis is made concerning how the resource trends may change in the ROI. If sufficient historical data were available, analyses of past records were used to quantify trends.

For the Crow Creek watershed ACS, nonproject-related impacts for future changes in population and land use are calculated using analytical models for potential increases in flood flows, wastewater discharges, and changes in raw water delivery.

Increases in flood flows were calculated by a series of methods. All require that drainage areas, drainage channels, and watershed characteristics (most importantly the degrees of imperviousness) be known.

Initial analysis calculated the change in impervious surface area to provide a guide on the percent increase in flood flow that might be anticipated. The Soil Conservation Service (SCS) has developed procedures to do this that are described in their Technical Release No. 55 (TR-55) (USDA 1975).

For watersheds where existing conditions would change to cause an increase in flood flow, additional analyses were made using more complex procedures. The synthetic unit hydrograph and hydrograph routing method based on travel time (M^{C}Cuen 1982) were used for the Crow Creek watershed area.

A qualitative analysis of erosion and sedimentation potential for all watersheds in the surface water ROI was made assuming typical erosion rates from existing types of land use and typical sediment delivery ratios.

The existing conditions in Crow Creek due to wastewater discharges are such that complex analytic techniques are not required to assess project-related impacts. Therefore, simple mass balance equations were used. The impact of new discharges due to land use and population changes without the project are calculated using similar techniques. Changes in raw water deliveries are calculated based on anticipated changes in raw water supply demands.

3.1.3.2 Proposed Action

The analytic techniques used to characterize future trends without the project are repeated with project-related changes in land use and population included for the Crow Creek watershed.

Qualitative analyses of project-related changes in the other ACSs are made using TR-55 methods for calculating stormwater impacts and mass balance equations for calculating the influence of wastewater discharges. Changes in baseline surface water flows could occur with increased wastewater discharges or with reductions in flow as a result of project water use or induced water use. These are qualitatively evaluated in the Crow Creek watershed.

The areas disturbed by construction at silos, road construction, communication system construction, and direct and indirect project construction in the ACSs watersheds are used in conjunction with typical erosion factors to qualitatively evaluate changes in erosion and sedimentation due to the project alternatives.

3.1.4 Groundwater Hydrology and Quality

The ACS for groundwater hydrology and quality includes the Crow Creek watershed ACS and areas in the immediate vicinity of municipal wells and wells, if any, developed for project construction in the surface water hydrology and quality ACSs in the DA.

Pumping from existing municipal wells to meet project-induced demand could influence hydrologic conditions in the Crow Creek watershed. Long-term use of well water for operations or use of wells to meet construction demands would likely not cause effects outside this ACS. Wastewater discharged by Cheyenne currently influences groundwater conditions by exfiltration from the streambed and this would continue with measurable impact only in Laramie County.

For the other ACSs, if wells are used to provide water for construction or to meet municipal demand, then the effects of the Proposed Action on the groundwater system are expected to occur only in the immediate vicinity of the wells. Because the location of project wells, if any, cannot be defined at present, a general overview of groundwater hydrology and quality in the DA is provided in this report.

3.1.4.1 Baseline Future - No Action Alternative

Groundwater hydrology and quality conditions are, as with surface water, determined by many factors. It is generally assumed that without the project, existing conditions and trends will continue into the future for the ACSs. Computer modeling of the groundwater system (Crist 1980) has provided an assessment of future trends for groundwater levels in the Crow Creek watershed ACS.

Where several wells are involved over an extensive area (e.g., the Cheyenne wellfield), regional models of groundwater flow are required to evaluate the impacts of various well pumping rates. An example of this type of model is the two-dimensional, finite-difference model developed by the U.S. Geological Survey (USGS) (Trescott et al. 1976). This model has been applied in the Crow Creek ACS (Crist 1980). A modification of this model was applied to the Cheyenne wellfield area as part of water supply development planning for this project (Earth Technology Corp. 1983b). Results of that modeling effort are summarized herein for purposes of impact assessment.

Groundwater quality changes due to wastewater discharges from Cheyenne were estimated with the simple mass balance developed earlier (Section 2.6.4.4) and projected effluent flows based on population changes without the project.

3.1.4.2 Proposed Action

Single well hydraulic equations are applied to predict impacts for alternative project well locations to indicate which areas need more extensive regional monitoring and/or modeling programs. This procedure will define areas of influence for proposed wells and the magnitude of water level declines that could be expected. Actual areas of influence and water level declines depend on site-specific conditions as determined by specific testing and monitoring programs for any new wells constructed.

When properties of an aquifer such as storativity, transmissivity, and boundary conditions are known, various equations of groundwater flow may be used to predict the impact of a pumping well on water levels in existing wells and on spring flow. Well pump tests need to be conducted to establish those aquifer properties. Single well equations of groundwater flow to be used in evaluating project impacts include the Theis equation, the Glover equation, the Hantush-Jacob equation, and the Weber technique (Bureau of Reclamation 1977).

Groundwater quality changes downstream of Cheyenne were again calculated with the mass balance developed; however, treated wastewater discharges were increased based on per capita flows and the projected population with the project.

3.2 Assumptions and Assumed Mitigations

3.2.1 Assumptions

Several assumptions are made related to water resource investigations, including:

- Water will be supplied by wells of up to 100 gallons per minute (gpm) capacity in the DA and/or by purchase or temporary use of water from existing wells.
- Nebraska Department of Environmental Control (NDEC), Nebraska Department of Health (NDH), and WDEQ permit requirements for sedimentation ponds, public water supplies, and facilities for wastewater treatment and disposal of wastewater will be followed.

- o Water acquisition efforts will follow Nebraska and Wyoming water law.

- o Wastewater discharge requirements of the Cheyenne Urban Area for discharge to Crow Creek will not change as a result of the project.

- o A monitoring program to document site-specific impacts on groundwater and surface water hydrology and quality will be implemented, as required.

- o The disturbed area used for analysis purposes is:

  - 0.5 acre per Launch Facility (LF);
  - about 3.3 acres per mile of gravel road upgraded;
  - about 2.4 acres per mile of communication cable;
  - about 130 acres at F.E. Warren AFB; and
  - developed acres for land use changes in the Cheyenne Urban Area.

3.2.2 Assumed Mitigations

Certain mitigations are part of standard Air Force construction practices or policy. For potential water resource impacts these will include:

- o Compensation to current water users who may be impacted during project construction;

- o Minimization of site disturbance and employment of proper revegetation techniques to reduce erosion potential;

- o Construction of stormwater detention and erosion control facilities to control increased surface runoff impacts; and

- o Development of water supply and waste disposal facilities for project-related facilities consistent with legal requirements.

The implementation of these standard mitigation measures will be assumed in evaluating the level and significance of impacts on the water resource system.

3.3 Level of Impact Definitions

For purposes of this project, the U.S. Air Force has established a two-level evaluation procedure to quantify and assess environmental impacts. The level of impact (LOI) within the ROI is determined according to predetermined criteria. The significance of the impact (i.e., sigrificant or not significant) is assessed according to predetermined criteria.

The Proposed Action may have a number of possible effects on the water resources of the ACSs. Environmental analysis consists of identifying various attributes of the water resource system, identifying the existing conditions of those attributes, and using qualitative or quantitative analytic techniques to determine the changes in existing conditions that would occur both with and without the proposed project. The changes in the water resource environment resulting from the proposed project are defined as the impact. Those changes can cause various levels of impact and may or may not be considered significant.

The level of impact refers to the magnitude and consequences of a change the project could make on a particular attribute of the water resource system for a particular area, whereas the significance of those impacts is a measure of how seriously those changes could affect environmental quality for human, animal, or plant life.

Significance is the measure of the importance of an impact. It reflects the interaction between LOI and the context in which the impact occurs. Context represents the various qualitative conditions present in the existing environment which operate to magnify or diminish the importance of the impact. The LOIs that would result from changes in the water resource system and the potential significance of those impacts are defined here for water resources.

For water resources in general, the LOI will be considered negligible, low, moderate, or high based on the following criteria:

- Negligible Impact - Would result in no easily measurable change in the projected baseline water resource system.
- Low Impact - Would result in a measurable change in the projected baseline water resource system that could require minor modification in operations.
- Moderate Impact - Would result in a measurable change in the projected baseline water resource system that would require minor modifications in operations or facilities.
- High Impact - Would result in a measurable change in the projected baseline water resource system that would require major changes in operations or facilities.

Impacts are considered short term if they would occur only during the period of project construction and long term if they would occur over the operational life of the project.

Specific criteria for defining the LOI are given in the following sections for water use and demand, constraints on water use, surface water hydrology and quality, and groundwater hydrology and quality.

3.3.1 Water Use and Demand

The Proposed Action would require water for its construction. It may also cause an induced water demand due to inmigrating construction workers and families, related construction, and other induced population drawn to an area because of the project or related construction. The following criteria provide the basis for defining the LOI of water use and demand on the water resource system:

- Negligible Impact - Would permit the increase in the amount of water required to meet project direct and induced demands to remain within the projected baseline available capacity of existing delivery systems and would require no change in system operation. No cost impact on existing users would occur.

- o Low Impact - Would permit the increase in the amount of water required to meet project direct and induced demands to remain within the projected baseline available capacity of existing delivery systems but would require minor increases in system operation to satisfy demands. No cost increases would be required to deliver the required amounts of water to existing water users.

- o Moderate Impact - Would increase the amount of water needed to meet project direct and induced demands so as to exceed the projected baseline available capacity of the existing delivery system. Capacity could be increased by construction of minor new facilities or by increases in system operation. The cost of the facilities to deliver the required amount of water could require minor cost changes for existing water users.

- o High Impact - Would increase the amount of water required to meet project direct and induced demands so as to exceed the projected baseline available capacity of existing delivery systems. Capacity could be increased by construction of major new facilities or major increases in system operation. Major water cost changes would result for existing users.

The increase in amount of water used, capacity of existing systems, and present cost of service are relatively easy-to-measure attributes of this water resource element. LOI is determined by changes to the existing system that will be needed to match water supply and water demand. Determination will be made on a project-specific basis.

3.3.2 Constraints on Water Use

The cost of, or difficulty in, meeting constraints relative to the acquisition and development of water resources or their protection will depend upon the area in which the project is located. Changes in constraints on water use are difficult to quantify and more qualitative evaluations are required. Some general criteria for assessing the LOI the project would have on existing individual water rights can be formulated (where right to use water is controlled).

Project-induced changes in the water resource system may require construction of new facilities or changes in operation of existing ones to comply with laws adopted to protect or manage the water resource system. The LOI will be determined based on the cost of such construction or changes.

- o Negligible Impact - Would require no acquisition of existing water rights.

- o Low Impact - Would require some acquisition of projected baseline existing water rights. Water rights would be relatively easy to acquire and acquisition would not affect the livelihoods of the former owners.

- o Moderate Impact - Would require acquisition of many projected baseline existing water rights. Water rights would be difficult to

acquire and would affect the livelihoods of the previous owners. Minor facilities may be needed in order to transport the water where it is needed.

- o High Impact - Would require more extensive acquisitions of projected baseline existing water rights than could be acquired in the area and major physical facilities would be needed to import water.

Existing water rights, laws, and regulations can be inventoried. Physical facilities proposed to avoid violations of existing law, or needed to develop new water sources, will be evaluated under other water resource elements using general water resource criteria for determining the LOI.

3.3.3 Surface Water Hydrology and Quality

The project could exert a number of possible impacts on surface water hydrology and quality, including an increase in the water erosion potential and associated water quality degradation, an increase in flood flows, an increase in wastewater discharge to streams, and an increase or decrease in surface water flow. Because of these varied possible impacts, defining the LOI requires multiple criteria. The LOI assigned will be the highest category for which any of these criteria would be satisfied.

- o Negligible Impact - Would result in no easily measurable projected baseline changes in water erosion, sedimentation potential, or associated water quality degradation, flood flows, wastewater discharge to streams, or changed surface water flow (whether increase or decrease).

- o Low Impact - Would result in small but measurable projected baseline changes in water erosion, sedimentation potential, and associated water quality degradation, flood flows, wastewater discharge to streams, or surface water flow that would require slight modifications in operations.

- o Moderate Impact - Would result in measurable projected baseline changes in water erosion, sedimentation potential, and associated water quality degradation, flood flows, wastewater discharge to streams, or surface water flow that would require modifications in operations or construction of minor new facilities.

- o High Impact - Would result in measurable projected baseline changes in water erosion, sedimentation potential, and associated water quality degradation, flood flows, wastewater discharge to streams, or surface water flow that would require major changes in operations or construction of major new facilities.

Each attribute of surface water hydrology and quality can be calculated using various analytic techniques to assign LOI.

3.3.4 Groundwater Hydrology and Quality

As with surface water hydrology and quality, there are several possible effects that the project could have on the groundwater hydrology and quality. These include reductions in water levels in existing wells or spring flow, degradation of groundwater quality from increased wastewater discharges, and degradation of groundwater quality due to mineral dissolution. Geologically related impacts, such as subsidence, that could be due to groundwater withdrawal, are addressed by the geologic resources task group. Once again, the LOI impact assigned will be the highest category for which any of the following criteria will be satisfied.

- Negligible Impact - Would result in no measurable projected baseline reductions in water levels in existing wells or spring flow, and no easily measurable degradation of groundwater quality from increased wastewater discharges.

- Low Impact - Would result in small but measurable projected baseline reductions in water levels in existing wells or spring flow, or small but measurable degradation of groundwater quality from increased wastewater discharges that would require slight modifications in operations.

- Moderate Impact - Would result in measurable projected baseline reductions in existing wells or spring flow, or a measurable degradation of groundwater quality from increased wastewater discharges that would require modifications in operations or construction of minor new facilities.

- High Impact - Would result in measurable projected baseline reductions in water levels in existing wells or in spring flow, or a substantial measurable degradation of groundwater quality from increased wastewater discharges that would require construction of major new facilities.

As in surface water hydrology and quality, each attribute of groundwater hydrology and quality can be calculated using various analytic techniques to assign LOI.

3.4 Significance Determination

For water resources, an impact is significant if: 1) it would violate laws or regulations adopted to protect or manage the water resource system; 2) it would endanger public health or safety by creating or worsening an adverse safety condition or health hazard; 3) it would reduce water availability to, or interfere with existing users creating a situation where a user would be unable to respond; 4) it would threaten or damage unique hydrologic characteristics of the area; and 5) it would change preproject hydrologic baseline conditions over an extensive area or period of time, so that resultant conditions are highly controversial, highly uncertain, or involve unique or unknown risks.

The determination of significance of impacts is applied on site-specific, local, and regional levels. For the purposes of the water resources

assessment, site-specific is defined as the location where direct construction activities are occurring and includes F.E. Warren AFB in the Crow Creek watershed and areas within 1,000 feet of LFs, Defense Access Roads (DAR), or cable paths. Local applies to impacts that would occur in a city jurisdiction or district surrounding an impacted area and includes the Cheyenne Urban Area in the Crow Creek watershed ACS and an area within 2 miles of LFs, DARs, cable paths, or potential project wells. Regional includes the entire watershed area in the ACSs in the ROI. If the anticipated water resource impacts do not meet any of these conditions for the defined area, then they would be considered not significant for that area. Further assessment guidance for each of the water resources elements is presented below.

3.4.1 Water Use and Demand

Evaluation of significance is based primarily on whether existing users suffer interference with, or reductions in available water service. Significance for the site-specific, local, or regional level depends on the relative number of users impacted by increases in the amount or costs of water required to meet project direct and induced demands and the location of those users.

3.4.2 Constraints on Water Use

Evaluation of significance is based primarily on laws and regulations adopted to protect or manage the water resources system. The site-specific, local, or regional significance depends on the number of water rights impacts and changes in constraints.

3.4.3 Surface Water Hydrology and Quality

Significance is primarily based on danger posed by impacts to public health and safety and changes in preproject hydrologic baseline conditions, although all significance criteria are evaluated in the final analysis. The site-specific, local, or regional significance depends on increases or decreases in surface flow, increases in erosion or flood potential, or degradation of water quality and how extensively those changes would affect human, animal, or plant life for each area. The impact is also considered significant if it would affect nationally recognized unique hydrologic features.

3.4.4 Groundwater Hydrology and Quality

Significance is primarily based on protection of public health and safety and on changes in preproject hydrologic baseline conditions, although all water resource significance criteria are evaluated.

The site-specific, local, or regional significance depends on water level reductions in existing wells, degradation of groundwater quality, and number of groundwater users for each area. The impact is also considered significant if it would affect nationally recognized hydrologic units (e.g., sole-source aquifers).

3.5 Environmental Consequences of the Proposed Action and No Action Alternative

This section describes the consequences of the Proposed Action and project alternatives and the future conditions without the project that would occur in the water resources ACSs for each water resource element. The analytic methods described in Section 3.1 are applied to predict changes from the existing conditions described in Section 2.6. An attempt has been made to quantify changes (i.e., acre-ft of increased water demand) in this section. Section 3.6 applies LOI and significance determination criteria to define the level and significance of these quantified changes. Where the analysis could not quantify change, a qualitative assessment was made based on professional judgment on changes likely to occur.

3.5.1 Water Use and Demand

Projection of water demand through the year 1991 encompasses the time period required for project construction and stabilization of the population to operational conditions. This section develops the baseline water demand with and without project deployment for the water use and demand ACS.

3.5.1.1 Baseline Future - No Action Alternative

The No Action Alternative description provides projections of future conditions in the ACS without project deployment for water use and demand.

3.5.1.1.1 Crow Creek Watershed

3.5.1.1.1.1 Municipal Water Use

The Cheyenne Urban Area is the only community in the Crow Creek watershed where the project would induce change. There are no other municipal water supplies in the watershed.

The projected population for the Cheyenne service area is shown in Table 3.5-1. Populations shown for the Cheyenne service area are for the water system service area, including South Cheyenne; this is based upon the assumption that 92.5 percent of the population in the Cheyenne Census Division plus all of F.E. Warren AFB will be served by the City. This is consistent with the Cheyenne Wastewater Facilities Plan assumption that "at least 92 percent" of the urban area population will be served by the City through the year 2005 (Banner Associates 1982).

Overall, nonindustrial water use for the Cheyenne service area is 180 gpcd. This figure has been used in projecting future water demands for municipal nonindustrial use. As discussed in Section 2.6.1.1, the industrial water demand in Cheyenne has remained relatively constant and is anticipated to increase only slightly at a rate of 100 acre-ft/yr over current use (CBPU 1983). Average industrial consumption in the Cheyenne service area from 1967 to 1978 was 2,220 acre-ft/yr, and in 1980 was 2,200 acre-ft. Industrial water use on F.E. Warren AFB would probably increase if the base population or employment increased. However, without project construction the base population is projected to remain at current levels.

Table 3.5-1

BASELINE POPULATION PROJECTIONS

	1983	1984	1985	1986	1987	1988	1989	1990	1991
Cheyenne Urban Area[1]	65,030	65,730	67,210	68,420	69,870	71,290	72,840	74,300	75,890
Cheyenne Service Area[2]	58,954	59,565	60,869	61,924	63,182	64,421	65,772	67,048	68,436
F. E. Warren AFB	3,630	3,630	3,630	3,630	3,630	3,630	3,630	3,630	3,630
Cheyenne Urban Fringe[3]	6,076	6,165	6,341	6,496	6,688	6,869	7,068	7,252	7,454
Rural Crow Creek Watershed[4]	980	1,012	1,061	1,097	1,146	1,193	1,239	1,286	1,334

Notes: 1 Includes Cheyenne Census Division, F. E. Warren AFB, and urban fringe area in Cheyenne East and West Divisions.
2 City of Cheyenne, South Cheyenne Water and Sewer District, and F. E. Warren AFB served population.
3 Within the Cheyenne urban area, but not in the Cheyenne Service area.
4 Outside the Cheyenne urban area.

Table 3.5-2 provides a year-by-year breakdown of projected water demands for the CBPU service area through the year 1991. Values represent the net amount of water that is delivered to the Cheyenne Urban Area through the raw water delivery system in the Crow Creek watershed and thus excludes reservoir evaporation and spillage losses.

Cheyenne's surface water supply capacity will be reduced temporarily by the construction of the Stage II water development system. Both Hog Park and Rob Roy reservoirs must be taken out of service while their capacities are enlarged. As a result, the expected transbasin diversions in 1983 and 1984 will be 2,400 and 5,500 acre-ft rather than the original Stage I capacity of 7,400 acre-ft (CBPU 1983). Shortages which will occur during the initial Stage II construction period are expected to be made up by pumping the municipal wellfields at rates greater than 2,000 acre-ft/yr, as well as by withdrawing water from storage in the Crow Creek reservoirs. Stage II will supply about 5,000 acre-ft/yr of additional water beginning in 1986, bringing the total transbasin diversion capacity to 12,400 acre-ft/yr, a value used by the Cheyenne Board of Public Utilities (CBPU) in water supply planning. The theoretical capacity of the Lake Owen to Crow Creek pipeline is 12,800 acre-ft/yr. A delivery rate of 12,400 acre-ft/yr allows for 3 percent down time for the pipeline or about 12 days per year. Other lower projections of supply consider more down time.

To calculate future water supplies available to meet future water demands, the schematic representation in Figure 2.6.4 is modified by the following assumptions:

- Water diversion from Douglas Creek will be 2,400 acre-ft in 1983; 5,500 acre-ft in 1984; 10,700 acre-ft in 1985; and up to 12,400 acre-ft in 1986 and later years. Evaporation losses from Lake Owen will continue at 205 acre-ft/yr. Transmission losses will increase proportional to existing conditions (257/6,600 or 3.9% of diversions).

- Total flow available for use from the Crow Creek watershed will be a function of inflow to the reservoirs. This value is obtained by deducting spillage and bypass (North Crow, 1,280; Bush Creek, 1,090; South Crow, 1,050; and Middle Crow, 1,110) and evaporation losses (North Crow, 250; South Crow, 10; and Middle Crow, 620) from total inflow consistent with current operating practice, and allocating Middle Crow losses proportionately to Middle Crow inflow and net inflow from Douglas Creek. Spilled water flows down Crow Creek and by infiltration helps to recharge the groundwater reservoir system.

  The calculation of Crow Creek yield differs from that historically used by the CBPU, but results in the same total supply. The CBPU method for 1976 to 1982 average conditions would have a Crow Creek watershed yield of 3,560 acre-ft/yr which equals North Crow yield (1,220 acre-ft/yr), plus Middle Crow yield which is calculated as the difference between the crystal meter reading (8,480 acre-ft/yr) and calculated Stage I inflow (6,140 acre-ft/yr or 2,340 acre-ft/yr. Total supply from surface water is calculated as Stage 1 inflow plus Crow Creek yield or 9,700 acre-ft/yr. This assigns all losses to the Middle Crow inflow. The method used in this analysis

Table 3.5-2

CHEYENNE BASELINE MUNICIPAL WATER SUPPLY AND DEMAND

Water Demand	1983	1984	1985	1986	1987	1988	1989	1990	1991
Industrial	2,500	2,600	2,700	2,800	2,900	3,000	3,100	3,200	3,300
Nonindustrial	11,890	12,010	12,270	12,490	12,740	12,990	13,260	13,520	13,800
TOTAL:	14,390	14,610	14,970	15,290	15,640	15,990	16,360	16,720	17,100
Water Supply									
Net Import from Douglas Creek[1]	1,510	4,120	8,850	10,430	10,430	10,430	10,430	10,430	10,430
Crow Creek Watershed[2]	4,150	4,520	4,790	4,840	4,840	4,840	4,840	4,840	4,840
Municipal Wells	2,000	2,000	2,000	2,000	2,000	2,000	2,000	2,000	2,000
TOTAL:	7,660	10,640	15,640	17,270	17,270	17,270	17,270	17,270	17,270
Supply-Demand	-6,730	-3,970	670	1,980	1,630	1,280	910	550	170

Note: 1 Net import downstream of reservoirs. Net inflow to reservoirs is 2,100 acre-ft in 1983, 5,080 acre-ft in 1984, 10,080 acre-ft in 1985, and 11,710 acre-ft in later years.

2 The occurrence of a one-in-ten-year drought in the area during the period would result in a reduction in the water supply from the Crow Creek Watershed of approximately 900 acre-ft. The result of this in any given year can be seen by reducing the supply by this and recalculating the "supply-demand," e.g., a drought in 1990 would result in a shortage of 350 acre-ft rather than a surplus of 550 acre-ft.

sets Middle Crow yield as Middle Crow inflow (4,070 acre-ft/yr) less losses from inflow calculated as a proportion of total inflow and losses

$$\frac{4,070}{4,070 + 6,140} \quad [620 + 470 + 640], \text{or } 690 \text{ acre-ft/ft.}$$

Middle Crow yield is thus 4,070 - 690, or 3,380 acre-ft/yr. This plus North Crow yield equals Crow Creek yield (4,600 acre-ft/yr). The total surface supply of 9,700 acre-ft/yr is Crow Creek yield plus net Stage I import, which is Stage I inflow (6,140 acre-ft/yr) less losses of 1,040 acre-ft/yr:

$$\frac{6,140}{4,070 + 6,140} \quad [620 + 470 + 640]$$

or 5,100 acre-ft/yr. Similar procedures were used for all future years.

- Groundwater supply to Cheyenne will be reduced to 2,000 acre-ft/yr (a gross pumpage rate of 2,500 acre-ft/yr to satisfy irrigation use contracts of about 500 acre-ft/yr) to satisfy concerns of the City of Cheyenne on lowering of water levels in the wellfields area at higher pumpage rates. A portion of the water supply planning for the project is evaluating the capability of the aquifers in the wellfield area to deliver water at rates in excess of 2,000 acre-ft/yr (Ertec Western, Inc. 1983d).

- Raw water losses in the raw water transmission lines in the Crow Creek watershed will remain negligible. An estimate of these losses of up to 24 acre-ft was made by Banner Associates (1983b).

- The occurrence of a 1-in-10-year drought would result in a reduction in available supply of about 900 acre-ft due to reduced runoff in Crow Creek watershed.

These assumptions are the basis of water supply values shown in Table 3.5-2.

The CBPU water supply capacity, including the Stage II Diversion, will be exceeded in about 1992. Following that time the City will be required to utilize groundwater at a rate higher than 2,000 acre-ft/yr, utilize more of the potentially available Crow Creek water supply, or build additional pipeline capacity to deliver additional water from Douglas Creek. At this time, there are no commitments to enlarge the pipeline capacity for the Stage II project (a proposed parallel 30-inch pipeline).

3.5.1.1.1.2 Industrial and Commercial Water Use

Industrial Water Demand. As mentioned previously, industries supplied by the CBPU are expected to increase their water consumption only slightly in the period through 1991. There are currently no known plans for large water-consuming industries to locate in the Cheyenne area. An allowance of 100 acre-ft/yr increase has been included herein to be conservative. Industries which provide their own water supplies are also not expected to show any significant increase in water consumption.

Commercial Water Demand. Commercial water users supplied by the City of Cheyenne will increase both in number and in the amount of water used as the Cheyenne population increases. It is expected that on a per capita basis commercial demand will be about 35 gpcd, the approximate existing rate for Cheyenne and South Cheyenne. Commercial water use is reported in the nonindustrial component of total Cheyenne demand in Table 3.5-2 and will vary from 2,310 acre-ft in 1983 to 2,680 acre-ft in 1991 in the Cheyenne Urban Area.

Commercial establishments with their own water supplies will be affected in several ways by growth in the Cheyenne area. As the City expands its corporate limits and services, some of these establishments will connect to the City water supply, and others, because of increased patronage, will increase their consumption somewhat. In addition, new establishments will open for business and develop their own water supplies; however, the overall change in water consumption, currently about 100 acre-ft/yr, will probably be quite small.

3.5.1.1.1.3 Agricultural Water Use

It is expected that the amount of water used for irrigation will increase only slightly through 1991. A large portion of the potentially irrigable land has currently been developed; in addition, the most heavily cultivated areas of Laramie County are located in the groundwater control area which place limitations on the capacity of wells which may be developed. As a result, it is projected that by 1991 a total of 50,000 acres will be irrigated with an annual water consumption of 70,700 acre-ft/yr. Whether or not irrigated lands will increase to the 50,000 acres assumed is unknown. Increased costs of groundwater pumping would tend to limit the increase in irrigation. Granting of water rights currently applied for but not granted would tend to increase irrigated acres. The value adopted provides for some increase in use for agricultural irrigation over the 46,400 acres irrigated in 1981 and the 48,200 acres irrigated in 1980. Because surface water supplies are fully allocated, it is expected that all new irrigation will be from groundwater. Projections are given in Table 3.5-3 for irrigation water use. Livestock water use in the Crow Creek watershed is expected to remain at current levels of about 280 acre-ft/yr.

The irrigated lands in the Crow Creek watershed have been estimated based on the projection for Laramie County and distribution of irrigated land in the county with results shown in Table 3.5-4.

Most of the irrigation in the Crow Creek watershed occurs downstream of Cheyenne with a considerable portion of surface irrigation water derived from

Table 3.5-3

PROJECTED IRRIGATION WATER USE - LARAMIE COUNTY

	Consumptive Use - 1981		Consumptive Use - 1990	
	acres	acre-ft	acres	acre-ft
Groundwater	26,000	37,000	30,000	42,700
Surface Water	20,000	28,000	20,000	28,000
TOTAL:	46,000	65,000	50,000	70,700

Table 3.5-4

PROJECTED IRRIGATION WATER USE - CROW CREEK WATERSHED

	Consumptive Use - 1981		Consumptive Use - 1990	
	acres	acre-ft	acres	acre-ft
Groundwater	6,600	9,400	7,300	10,400
Surface Water	4,700	7,000	4,700	7,000
TOTAL:	11,300	16,400	12,000	17,400

wastewater effluent from Cheyenne. The Crow Creek water user group has initiated a study through the Wyoming Water Development Commission to evaluate increasing water supplies to allow an increase in irrigation (Water Development Commission 1983) which could alter the above projections.

3.5.1.1.1.4 F.E. Warren AFB Water Use

Water use by F.E. Warren AFB is expected to remain at current levels through 1991 without the project. Base population is expected to remain stable at 3,630 and no shift in types of activities carried out at the base would be expected. There are currently some proposals to reduce water consumption on the base by reclaiming wastewater for use in irrigating the golf course and parade grounds (Baker, Sweeney and Associates 1981). There are, however, some institutional problems which could preclude this from being implemented in the near future, i.e., water users downstream of the city of Cheyenne have established a use for the city's effluent and reduction in effluent discharge could infringe upon these water rights.

3.5.1.1.1.5 Other Water Uses

Rural domestic water used by individuals living in unincorporated areas of the Crow Creek watershed will also increase through the year 1991. Table 3.5-1 shows the population for these areas. It is projected that consumption from private wells will be 100 gpcd for these areas. The total demand in these areas will rise from 680 acre-ft in 1980 to about 980 acre-ft/yr in 1991 for population outside the CBPU service area.

Table 3.5-5 presents a summary of the baseline water use for the Crow Creek watershed ACS.

3.5.1.1.2 Other Areas in the Region of Influence

In the No Action Alternative, there are only slight increases in water use anticipated in the other water use and demand ACSs. This section will discuss each component of water use that has been previously discussed.

3.5.1.1.2.1 Municipal Water Use

Only eight communities in the ACSs are anticipated to have increases in water demands due to the project. The baseline water demands for these communities for the No Action Alternative are shown in Table 3.5-6. Existing water supplies will be adequate for all these communities with the No Action Alternative.

Because only total water supply figures are available for these communities, it is difficult to assess the true residential component of water use for projection purposes. For this analysis, a value of 250 gpcd was selected for population in excess of the current population. The average per capita use for these communities is currently about 400 gpcd; however, this value also includes existing industrial use. Because no plans exist for major industrial expansion in any of these communities, a somewhat lower value seems appropriate for water use projections. It was felt that 250 gpcd is a conservative yet reasonably representative value to use for this purpose. The value is also the average of all municipalities in the ROI potentially impacted by the Proposed Action.

Table 3.5-5

BASELINE WATER USE - CROW CREEK WATERSHED AREA OF CONCENTRATED STUDY
ACRE-FT/YR

	1983	1984	1985	1986	1987	1988	1989	1990	1991
Municipal[1]	14,390	14,610	14,970	15,290	15,640	15,990	16,360	16,720	17,100
Industrial/Commercial[2]	970	970	970	970	970	970	970	970	970
Irrigation	16,630	16,740	16,850	16,960	17,070	17,180	17,290	17,400	17,510
Livestock	280	280	280	280	280	280	280	280	280
Domestic (Nonmunicipal)[3]	1,040	1,050	1,080	1,100	1,130	1,150	1,180	1,210	1,230
TOTAL:	33,310	33,650	34,150	34,600	35,090	35,570	36,080	36,580	37,090

Notes: 1 Includes Cheyenne, South Cheyenne and F. E. Warren AFB.
2 Includes only self-supplied users.
3 Includes private wells in South Cheyenne at 250 acre-ft/yr.

Table 3.5-6

BASELINE MUNICIPAL WATER USE - OTHER AREAS OF CONCENTRATED STUDY
ACRE-FT/YR

	1983	1984	1985	1986	1987	1988	1989	1990
Albin	71	71	71	72	72	72	73	73
Pine Bluffs	524	528	532	536	540	544	547	552
Wheatland	2,120	2,150	2,170	2,200	2,230	2,270	2,300	2,340
Chugwater	37	40	43	46	49	51	54	57
Torrington	4,850	4,870	4,890	4,940	5,000	5,050	5,110	5,170
Kimball	1,010	1,010	1,010	1,010	1,010	1,020	1,020	1,020
Gering	3,300	3,370	3,450	3,530	3,610	3,690	3,780	3,860
Scottsbluff	4,490	4,520	4,540	4,570	4,590	4,610	4,630	4,650

Note: Only communities with project-induced population growth shown.

3.5.1.1.2.2 Industrial and Commercial Water Use

There are currently no plans for new, self-supplied industrial water users to locate in this area, nor are there plans for major expansion of existing users. Therefore, it has been assumed that industrial and commercial use will remain constant throughout the 1983 to 1990 period. Any increase is included in the 250 gpcd value used in projections.

3.5.1.1.2.3 Agricultural Water Use

There does not appear to be an increasing trend in livestock production in the ROI and for this reason it has been assumed that livestock water use will remain at current levels.

There are, however, definite trends toward increased irrigation. This is particularly true in Nebraska where acquisition of water for irrigation is relatively simple both physically and administratively. The Nebraska Natural Resources Commission has projected groundwater irrigation growth rates on a county-by-county basis and are shown in Table 3.5-7. It is expected that the increase in groundwater irrigation in Wyoming will be somewhat less due to the presence of two groundwater control areas in the Wyoming portion of the ROI. Generally speaking, surface waters in the area are fully appropriated, thereby limiting the amount of additional land that can be irrigated with surface water.

Acreage irrigated with surface water has been assumed to remain constant through 1990 for all counties except Goshen. The recent construction of Greyrocks Reservoir has made some additional surface water available for irrigation use in Goshen County. In addition, there were approximately 95,000 acres within surface water irrigation districts in Goshen County in 1977 although all of these are not irrigated (Oblinger-Smith Corp. 1977). It is assumed that the full 95,000 acres will be irrigated by 1990.

3.5.1.1.2.4 Other Water Uses

Future nonmunicipal domestic water use in other ACSs is projected to remain relatively constant through 1990 with the No Action Alternative. Water use at the existing Minuteman sites is expected to remain at current levels of about 12 acre-ft/yr.

Table 3.5-8 shows the projected water use for the No Action Alternative. Overall use is expected to increase by about 14 percent with the largest portion of this attributable to agricultural irrigation.

3.5.1.2 Proposed Action

Water would be required for project construction and operation, as well as to satisfy demands induced in the ROI by inmigrating workers, operations personnel, and their dependents. Construction water would be required primarily for DAR upgrades; this water would be used for road bed compaction, revegetation, and bridge and culvert modification or replacement. Additional construction water use would occur at F.E. Warren AFB and 100 individual LFs and 10 Launch Control Facilities (LCFs). Induced demands would primarily affect the Cheyenne Urban Area. A continuing operation demand would also occur in the Cheyenne Urban Area.

Table 3.5-7

BASELINE IRRIGATION WATER USE - OTHER ACSs
(acre-ft/yr)

	1983	1984	1985	1986	1987	1988	1989	1990
Banner County								
Groundwater	27,900	28,900	29,800	31,100	32,400	33,700	35,000	36,300
Surface Water	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000
TOTAL:	28,900	29,900	30,800	32,100	33,400	34,700	36,000	37,300
Kimball County								
Groundwater	37,200	40,200	43,300	44,900	46,400	47,900	49,500	51,000
Surface Water	1,000	1,000	1,000	1,000	1,000	1,000	1,000	1,000
TOTAL:	38,200	41,200	44,300	45,900	47,400	48,900	50,500	52,000
Scotts Bluff County								
Groundwater	61,400	63,600	65,900	68,000	70,100	72,300	74,400	76,500
Surface Water	282,000	282,000	282,000	282,000	282,000	282,000	282,000	282,000
TOTAL:	343,400	345,600	347,900	350,000	352,100	354,300	356,400	358,500
Nebraska, total								
Groundwater	126,500	132,700	139,000	144,000	148,900	153,900	158,900	163,800
Surface Water	284,000	284,000	284,000	284,000	284,000	284,000	284,000	284,000
TOTAL:	410,500	416,700	423,000	428,000	432,900	437,900	442,900	447,800
Goshen County								
Groundwater	39,700	40,500	41,400	42,200	43,100	43,900	44,700	45,600
Surface Water	119,000	122,900	125,800	128,800	131,800	134,700	137,700	140,600
TOTAL:	158,700	163,400	167,200	171,000	174,900	178,600	182,400	186,200

Table 3.5-7 Continued, Page 2 of 2
BASELINE IRRIGATION WATER USE - OTHER ACSs

	1983	1984	1985	1986	1987	1988	1989	1990
Laramie County[1]								
Groundwater	28,640	29,170	29,690	30,210	30,730	31,260	31,780	32,300
Surface Water	21,000	21,000	21,000	21,000	21,000	21,000	21,000	21,000
TOTAL:	49,640	50,170	50,690	51,210	51,730	52,260	52,780	54,300
Platte County								
Groundwater	48,200	49,200	50,300	51,400	52,500	53,600	54,700	55,800
Surface Water	20,000	20,000	20,000	20,000	20,000	20,000	20,000	20,000
TOTAL:	68,200	69,200	70,300	71,400	72,500	73,600	74,700	75,800
Wyoming, total								
Groundwater	116,500	118,900	121,400	123,800	126,300	128,800	131,200	133,700
Surface Water	160,000	163,900	166,800	169,800	172,800	175,700	178,700	181,600
TOTAL:	276,500	282,800	288,200	293,600	299,100	304,500	309,900	315,300
Total, Wyoming and Nebraska								
Groundwater	243,000	251,600	260,400	267,800	275,200	282,700	290,100	297,500
Surface Water	444,000	447,900	450,800	453,800	456,800	459,700	462,700	465,600
TOTAL:	687,000	699,500	711,200	721,600	732,000	742,400	752,800	763,100

Note: 1 Excludes Crow Creek watershed.

Table 3.5-8

BASELINE WATER USE IN OTHER AREAS OF CONCENTRATED STUDY
ACRE-FT/YR

	1980	1984	1985	1986	1987	1988	1989	1990
Municipal	20,060	20,200	20,350	20,550	20,750	20,950	21,160	21,370
Industrial/Commercial	44,100	44,100	44,100	44,100	44,100	44,100	44,100	44,100
Irrigation	664,600	699,500	711,200	721,600	732,000	742,400	752,800	763,100
Livestock	7,500	7,500	7,500	7,500	7,500	7,500	7,500	7,500
Domestic (Nonmunicipal)	2,790	2,510	2,540	2,570	2,590	2,620	2,650	2,680
TOTAL (ROUNDED):	739,000	774,000	786,000	796,000	807,000	818,000	828,000	839,000

3.5.1.2.1 Construction Water Requirements

3.5.1.2.1.1 Deployment Area

Construction would occur in the 319th and 400th Squadrons. The present worst case estimate involves resurface work on 340 miles of county and Air Force gravel roads. In addition, some 305 miles of existing paved surface may be repaved. Water demands for gravel road upgrade (whether surfacing Option A or Option B) will be about the same. Road repair and upgrading would account for 409 acre-ft of the total construction water requirement (this includes a 100-percent contingency). The road upgrading would occur in 1984 to 1987 with a peak water requirement of about 168 acre-ft in 1986. Approximately 270 acre-ft of the road construction water would be required in Wyoming; the remaining 139 acre-ft would be used in Nebraska. An additional 40 acre-ft of water will be used throughout the DA, primarily for access improvements and other work at the LFs and LCFs. Table 3.5-9 shows the total construction water requirements for the project.

Table 3.5-9
CONSTRUCTION WATER REQUIREMENTS
ACRE-FT

	1984	1985	1986	1987	1988	1989	Total
F.E. Warren AFB	13	49	5	--	--	--	67
Deployment Area							
Nebraska	--	--	56	88	5	--	149
Wyoming	8	160	123	6	3	--	300
TOTAL:	21	209	184	94	8	--	516

There will be continuing operational demand of 12 acre-ft/yr in the DA. This is identical to water use for existing Minuteman installations in the DA.

3.5.1.2.1.2 Construction Facilities

Water would be required for aggregate washing and concrete batching; this use would occur at facilities located some distance from the actual construction sites. The location of these facilities has not been determined at this time; however, approximately 60 acre-ft of water would be required for these purposes regardless of the location. In addition, some water would be required for revegetation for these areas.

The Proposed Action includes construction dispatch stations at F.E. Warren AFB, Chugwater, and Kimball. These areas would require development of a limited water supply for domestic use by workers; the quantity of water to be used is not expected to be significant (25 office workers would use about 0.5 acre-ft/yr) and is included in the contingencies shown in Table 2.2-1.

3.5.1.2.1.3 F.E. Warren AFB

Some construction of additional facilities on the base would be required for the project. This would consist of additional control and maintenance facilities, as well as facilities for the use in assembly of the project components prior to their delivery to the LFs. This work would occur in 1984 to 1986 and would require 67 acre-ft. This water would be supplied by the CBPU water system.

3.5.1.2.2 Induced Demand

Construction workers, military personnel, and their dependents moving to the ROI for project construction and operation would exert a demand upon the water supplies of all ACSs. The majority of this demand would be supplied by the CBPU, although several other communities in the ROI will experience increased demands.

3.5.1.2.2.1 Cheyenne Urban Area

Construction of the Proposed Action would result in an increase in the Cheyenne area population. In addition, some construction workers are expected to drive to Cheyenne on a temporary basis, staying in motels or other temporary quarters and returning to their permanent residences occasionally. Water use for these individuals as well as unsuccessful job-seekers (transients) is projected to be about 100 gpcd on an average daily basis.

In determining increased water demands, it has been assumed that new residents would have a nonindustrial water use of 180 gpcd; this includes residential, commercial, and civic uses as discussed in Section 2.6.1.1.1.1. Industrial water use would increase by 100 acre-ft/yr as in the baseline condition. The impact of the total increase in water use in Cheyenne is shown in Table 3.5-10.

The maximum increase in demand of 571 acre-ft/yr would occur in 1987. This represents an increase of 3.7 percent over the demand in the No Action Alternative. The long-term increase in demand due to the project is 250 acre-ft/yr. This is an increase in demand of 1.5 percent in 1991. The ability of Cheyenne's water system to supply the water necessary to satisfy project-induced demand is somewhat limited in the initial project years. This is shown in Table 3.5-11. Because of the construction of the Stage II diversion facilities, the CBPU is currently expecting to have insufficient supply in 1983 and 1984; these shortages will be made up by increasing groundwater pumping rates or withdrawing water from storage in the Crow Creek watershed. The Proposed Action would have a slight impact on these shortages with an approximate increase of only about 70 acre-ft in 1984. Following completion of the Stage II Diversion Project the CBPU system will have adequate water under nondrought conditions to meet the demand of its service area with the project through 1990. After 1984 the impact of the project is to reduce the amount of surplus water available for rebuilding the storage in the reservoirs. The long-term operating demand of 250 acre-ft/yr would have the effect of moving up by 1 year the construction of a second pipeline from Lake Owen. With the No Action Alternative, this would be required about 1992; with the Proposed Action, this additional capacity would be required about 1991. Figure 3.5-1 shows the impact of the project on the CBPU water supply system.

Table 3.5-10

PROJECT DEMANDS ON CHEYENNE WATER SUPPLY

	1984	1985	1986	1987	1988	1989	1990	1991
Inmigrants (Number)								
Permanent	206	1,026	2,047	2,363	2,156	2,092	1,044	925
Transient	69	324	228	262	294	233	156	-
Commuters	25	75	125	25	<10	<10	-	-
Water Demand (acre-ft)								
Induced Water Demand	53	251	453	508	469	449	228	187
F.E. Warren AFB Operation Demand	-	7	63	63	63	63	63	63
F.E. Warren AFB Construction Use	13	49	5	-	-	-	-	-
TOTAL:	66	307	521	571	531	512	291	250
Portion Used at F. E. Warren AFB	17	81	96	92	91	91	79	76

Note: 1 Based upon 25 gpcd for workers commuting to the Base.

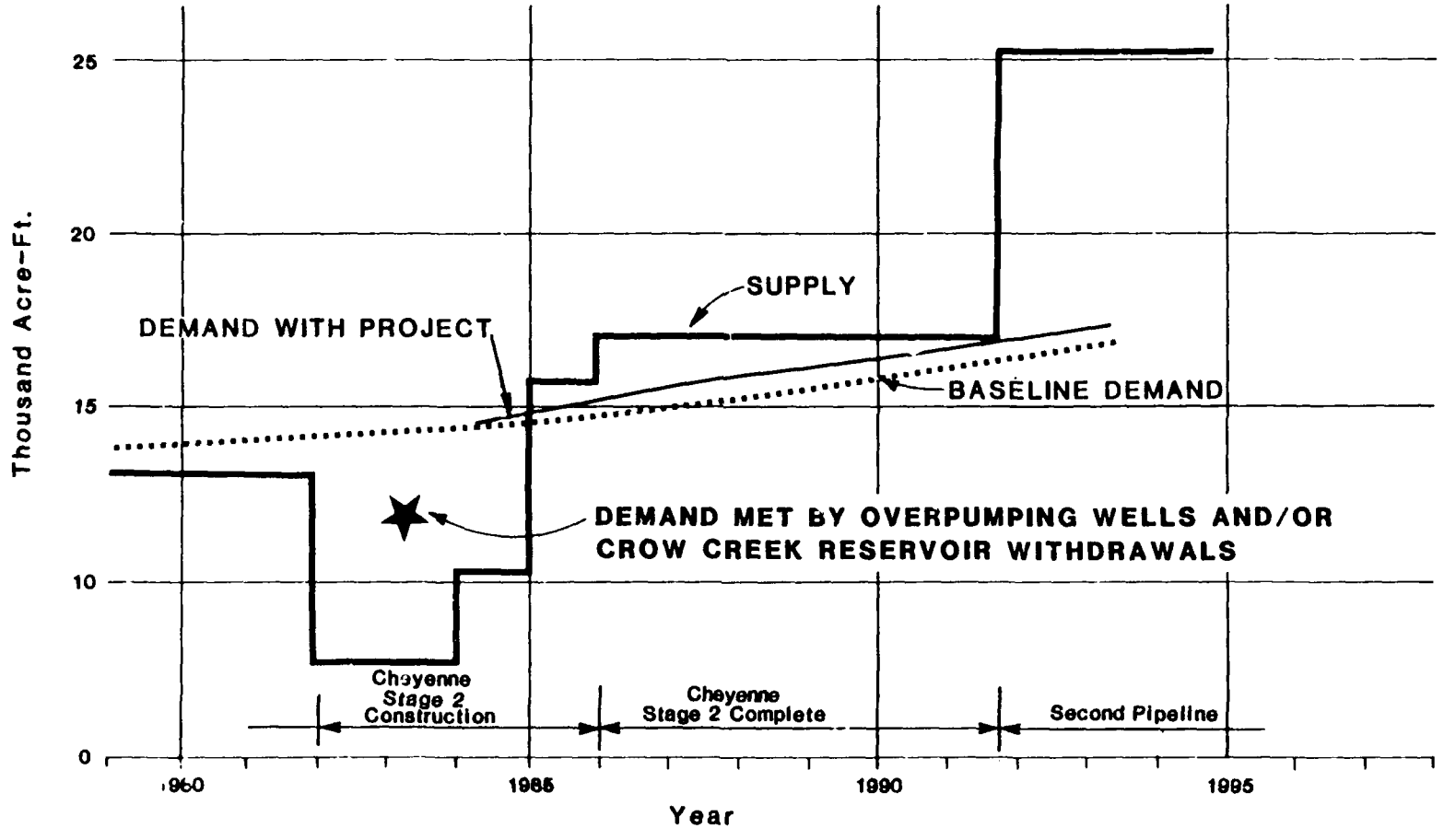
Table 3.5-11

CHEYENNE MUNICIPAL WATER DEMANDS - PROPOSED ACTION
ACRE-FT/YR

	1983	1984	1985	1986	1987	1988	1989	1990	1991
Water Demand									
Baseline	14,390	14,610	14,970	15,290	15,640	15,990	16,360	16,720	17,100
With Project	14,390	14,680	15,280	15,810	16,210	16,520	16,870	17,010	17,350
Water Supply									
Stage I & II Import	1,510	4,120	8,850	10,430	10,430	10,430	10,430	10,430	10,430
Crow Creek Watershed[1]	4,150	4,520	4,790	4,840	4,840	4,840	4,840	4,840	4,840
Municipal Wells	2,000	2,000	2,000	2,000	2,000	2,000	2,000	2,000	2,000
TOTAL:	7,660	10,640	15,640	17,270	17,270	17,270	17,270	17,270	17,270
Supply - Demand									
Baseline	-6,730	-3,970	670	1,980	1,630	1,280	910	550	170
With Project	-6,730	-4,040	360	1,460	1,060	750	400	260	-80

Note: With project water demands rounded to the nearest 10 acre-ft.

1 The occurrence of a one-in-ten-year drought during the period would result in a reduced water supply due to less runoff in the Crow Creek Watershed. It is estimated that this reduction would be approximately 900 acre-ft. The effect of this can be seen by reducing the "Supply-Demand" value accordingly, e.g., a drought in 1989 would result in a 10 acre-ft surplus in the Baseline, and a 500 acre-ft shortage with the project.



PROJECT IMPACT ON CHEYENNE
URBAN WATER SUPPLY

FIGURE NO.3.5-1

In order to meet the demands shown in Table 3.5-11, the CBPU has a range of possible operational alternatives. Because of the shortage caused by Stage II constructions in 1983 and 1984 the water supply system will have a reduced factor of safety, i.e., the wells will have been overpumped for two consecutive years and the reservoir storage will be reduced. Because of this the CBPU desires to refill its storage reservoirs and reduce pumpage of its wellfields as much as possible while meeting demands. An "informal" goal of the CBPU is to maintain at least 6,000 acre-ft of water in the three Crow Creek reservoirs (Granite Springs, Crystal Lake, and North Crow). At the end of 1983 these reservoirs are projected to contain about 5,600 acre-ft of water according to the Director of the CBPU. One possible operational alternative that meets project demands is presented in Table 3.5-12. As shown in this table, wells could be operated at the desired level of 2,000 acre-ft/yr and the reservoirs could be refilled to the desired level of 6,000 acre-ft by 1986. In addition, the reservoirs would not drop appreciably below 1983 levels. The total increase in groundwater withdrawals over baseline conditions with this operational alternative is 70 acre-ft.

In addition to average conditions, the possibility of a drought year occurring during the project period must be considered. As discussed earlier the impact of a 1-in-10 year drought is to reduce the CBPU water supply by about 900 acre-ft. The effect of this on the operational alternative presented in Table 3.5-12 is to reduce the supply-demand value for the drought year by 900. This shortage would be made up by increasing pumping or by decreasing water in storage. As shown in the table, enough storage exists in any of the project years to meet this shortage. However, if the drought occurred in 1984 or 1985, the resulting end of year storage would be less than the desired 6,000 acre-ft if this were done, although excess supplies in subsequent years would replenish the storage. Historically, voluntary water conservation measures implemented during drought conditions have helped to reduce water demands.

3.5.1.2.2.2 F.E. Warren AFB

F.E. Warren AFB receives its entire water supply from the CBPU and will continue in this manner throughout the project period. The increased water requirements for construction and operation will be supplied by Cheyenne's system. Several new buildings and employees will be located at the base resulting in a continuing operational demand of 7 acre-ft in 1985 and 63 acre-ft in 1986 and later years. The increased operational demand at F.E. Warren AFB represents about 27 percent of the long-term increased demand on the CBPU system.

3.5.1.2.2.3 Other Areas

Because the entire inmigrant population in the Crow Creek watershed is projected to be served by Cheyenne, there would be no other water supply impacts anticipated in the watershed.

The communities of Pine Bluffs, Torrington, Wheatland, Kimball, Gering, and Scottsbluff are projected to have some population increase due to the project. The increased water demands associated with these increases are shown in Table 3.5-13. There are no long-term population increases projected in any of these communities due to the project. All of these communities currently have sufficient water supplies to provide these demands.

Table 3.5-12

EFFECT OF PROPOSED ACTION ON CBPU WATER SUPPLY SYSTEM OPERATION

	1983	1984	1985	1986	1987	1988	1989	1990	1991
Supply-Demand (acre-ft)	-6,730	-4,040	360	1,460	1,060	750	400	260	-80
Groundwater Over-Pumping (acre-ft)[1]	2,500	4,000	0	0	0	0	0	0	0
Change in Storage (acre-ft)	-4,230	-40	+360	+1,460	+1,060	+750	+400	+260	-80
End of Year Storage (acre-ft)	5,600	5,560	5,920	7,380	8,440	9,190	9,590	9,850	9,770
Increase in Pumping Required Over the Baseline Condition (acre-ft)	0	70	0	0	0	0	0	0	0

Note: 1 In excess of 2,000 acre-ft.

Table 3.5-13

MUNICIPAL WATER USE IMPACTS IN OTHER ACSs

PROJECT-INDUCED INCREASE
(acre-ft/yr)

	1984	1985	1986	1987	1988	1989	1990	Total
Pine Bluffs	0	0	3	0	42	0	0	45
Albin	0	1	1	1	0	0	0	3
Torrington	0	0	0	59	0	0	0	59
Chugwater	0	15	15	14	0	0	0	44
Wheatland	0	38	121	52	0	0	0	211
Kimball	0	0	0	8	22	80	0	110
Gering	0	0	9	9	31	23	0	72
Scottsbluff	0	1	18	17	61	45	0	142
TOTAL:	0	55	167	160	156	148	0	686

TOTAL WATER USE WITH PROJECT
(acre-ft/yr)

	1984	1985	1986	1987	1988	1989	1990	System Capacity
Pine Bluffs	528	532	539	540	586	547	552	2,745
Albin	71	72	73	73	72	73	73	200
Torrington	4870	4890	4940	5059	5050	5110	5170	8,450
Chugwater	40	58	61	63	51	54	57	155
Wheatland	2150	2208	2321	2282	2270	2300	2340	3,800
Kimball	1010	1010	1010	1018	1042	1100	1020	1,805
Gering	3370	3450	3539	3619	3721	3803	3860	6,500
Scottsbluff	4520	4541	4588	4607	4671	4675	4650	11,035

Other types of water use are not expected to change as a result of the project. This includes privately supplied commercial and industrial facilities and agricultural water use.

3.5.1.2.3 Demand Reduction Opportunities

The water use figures presented in this section reflect a minimal amount of water reuse and conservation practices; recirculation of concrete aggregate washwater is one of the few methods incorporated in these numbers. As a result, it is possible that the amount of water required could be reduced somewhat by utilizing water conservation practices. Some of these include:

- Reuse of treated wastewater for construction purposes, primarily revegetation at F.E. Warren AFB;
- Demand reduction at F.E. Warren AFB by reusing wastewater for irrigation purposes; and
- Accelerated implementation of water conservation practices in the city of Cheyenne.

These and other potential methods for reducing water demand will be discussed in Section 3.7 along with other mitigation measures.

3.5.1.2.4 Water Development Plans

The project water demands described previously require development of water supplies for project construction and long-term operation, and to support construction and operational personnel and their dependents. Ertec Western, Inc., under separate contract to the Air Force, has formulated water-supply development plans (structured procedures) which describe alternative water sources and define the activities necessary to obtain and develop regional water supplies by April 1984, the projected initial construction date. The following sections summarize those plans.

3.5.1.2.4.1 Cheyenne Urban Area

Description. Water will be required in the Cheyenne Urban Area to meet increased demands for construction and operational personnel, as well as for a small amount of construction water at F.E. Warren AFB in 1984 to 1986. There are a number of alternatives for providing the increased water; generally they can be described as increased or more efficient use of existing water sources or development of new raw water sources. The potential sources of increased water supply are described below.

- Increased pumpage of municipal wellfields - In the period from 1976 to 1982 Cheyenne has used an average of about 3,320 acre-ft/yr from its municipal wellfields; the desired usage is a maximum of 2,000 acre-ft/yr. In order to meet shortages until the completion of construction of the Stage II Diversion expansion, it is expected that these wells will be used to provide between 4,000 and 7,000 acre-ft/yr in 1983 and 1984. The wellfields could then be used at the desired rate of 2,000 acre-ft/yr or less starting in 1985.

- Use of Stage II Diversion facilities - The current Stage II construction project has temporarily reduced Cheyenne's water system capacity so that shortages are projected for 1983 and 1984; however, upon its completion in 1985 the CBPU will have raw water in excess of demands; this will allow the return of groundwater pumping rates to desired levels and the refilling of storage reservoirs. This excess "import" water available from the Stage II project could be used to meet project-induced demands. If this were done, excess water would still be available for refilling reservoirs, however, at a lower rate than in the baseline condition.

  Early construction of additional Stage II pipeline capacity would provide additional water to the Cheyenne area; however, this additional water would not be required to meet project demands. Current plans call for construction of this pipeline in the early 1990s. The long-term effect of the project is to move up the needed date for completion of this pipeline by 1 year .

- Increased supply from Crow Creek watershed - The possibility of constructing increased reservoir capacity in the Crow Creek watershed has been evaluated and rejected as not cost-effective (Banner 1983b). However, it is possible that the yield from this watershed could be increased with modifications and/or improvements to existing facilities. The use of snow fences to increase watershed yields has been studied in the Crow Creek Basin (Tabler 1971). This study indicated that runoff from the South Fork of Crow Creek was one-third less than that from the Middle Fork, primarily due to lower snow capture due to topographic and vegetative differences. Based upon data from this study, it is possible that the yield from Crow Creek could be increased by about 200 acre-ft/yr by placing snow fences in portions of the North, Middle, and South forks of Crow Creek.

  The entire Crow Creek watershed supply system is not currently used. The Brush Creek Diversion, located on a tributary to North Crow Creek, consists of an infiltration diversion along the creek, as well as a direct diversion. The direct diversion is subject to periods of high turbidity and is not used during these times. The infiltration diversion, which is not subject to turbidity problems, is controlled by the same valve as the direct diversion; therefore, it is not used during high runoff periods (Banner 1983a). A modification of this valving arrangement would increase the amount of water available from this portion of the system. In addition, the construction of a small holding reservoir on Brush Creek would likely decrease turbidity problems and allow additional use of these facilities.

  The South Crow Diversion consists of a small dam (storage capacity, 12 acre-ft) and a diversion pipeline to the Middle Crow Creek pipeline. This diversion is currently not used because of turbidity problems (Banner 1983b). The estimated potential yield of South Crow Creek above the diversion is almost 1,100 acre-ft/yr, although the size of the diversion dam limits the amount of this that could

be captured. It is, however, possible that structural or operational changes could be made to allow some use of the South Crow Diversion by decreasing turbidity problems and/or increasing yield.

- Wastewater reuse - F.E. Warren AFB currently purchases all its water from the CBPU; this includes approximately 225 acre-ft/yr used for irrigation of the golf course and parade grounds at the base. With the induced demands associated with the Proposed Action it is projected that Cheyenne's wastewater flows would be sufficient to allow construction and operation of a wastewater reuse system at F.E. Warren AFB by 1985 that could provide the entire annual irrigation demand of 225 acre-ft. This reduction in demand would nearly offset the projected long-term project demand of 250 acre-ft and in each year after 1985 would allow the CBPU additional water for refilling reservoirs.

- Water conservation - Cheyenne is currently in the process of implementing a voluntary water conservation program involving public education in conjunction with rate increases and a restructuring of the rate schedule to encourage conservation. The new rates are being implemented gradually and are scheduled to be in place by July 1, 1985. While it is not possible to quantify an actual use reduction due to this, it is likely that this program will have some impact on consumption rates. A reduction in projected baseline water use of about 3.7 percent in 1987 would entirely offset the project-induced demand in the year of peak impact.

- Development of the Casper (Paleozoic) aquifer - The water yield potential and water quality of the Casper aquifer on the west side of the Laramie Range are relatively well defined. However, development of this potential water supply source in this general location would require an extensive conveyance (pipeline/pumping) system to deliver water to Crow Creek and the existing Cheyenne municipal water supply system. Consequently, consideration of development of the Casper aquifer to the west of the Laramie Range for use in the Cheyenne area is probably not a viable alternative.

  The water-yield potential and water quality of the Casper aquifer on the east side of the Laramie Range are essentially undefined. Based on general hydrogeologic considerations, it is generally assumed to be a viable although limited water supply source. The Casper aquifer does not offer a near-term water supply source because of the length of time required for evaluation, exploration, and development. However, if development of significant quantities of groundwater is feasible in close proximity to Cheyenne, it could provide a long-term water supply source which could augment or supplant a near-term source developed or used to meet initial project demands for the Cheyenne Urban Area. However, preliminary evaluations indicate that development of large quantities of water in this area is unlikely and the yields probably would not exceed several hundred acre-ft/yr (Earth Technology Corp. 1983a).

- Development of the Ogallala/Arikaree aquifers - Development of the shallow Tertiary (Ogallala and Arikaree) aquifers in an area approximately 15 miles northeast of Cheyenne has been considered as an alternative in previous water supply planning for Cheyenne (Banner and Associates 1961, Wyoming Water Resources Research Institute 1979). The saturated thickness of the Ogallala/Arikaree aquifers in this area exceeds 400 feet and potential yields may exceed 1,000 gpm. The alternative has been historically rejected because of the cost of the conveyance system required, the potential for impact to flow in Horse Creek, and the fact that the area is within the Laramie County groundwater control area. It is likely that these factors will continue to limit the viability of this alternative in the present evaluation, although with the amounts of water required to meet project-induced demands the effects would likely be less than those previously determined for the use of this system for a long-term supply for Cheyenne.

- Purchase of existing water rights - To augment existing water supplies, it may be feasible to purchase or lease existing water rights to meet short-term project-induced demands. These existing water supply sources (wells or surface water diversion) could then be added as input to the municipal water system. Because of proximity, the time required to physically add supply sources may be minimal.

  Records obtained from the Wyoming State Engineers Office indicate that there are less than 2,000 acre-ft/yr of groundwater irrigation rights in the 6 townships encompassing the Crow Creek watershed to the west of Cheyenne (T13 and 14N; R67; 68 and 69W). Since only a portion of these rights may be legally available for purchase, the magnitude suggests that enough water to meet the estimated project demands for the Cheyenne area could not be obtained in the area.

Effects on Groundwater Hydrology and Quality. The effect of the various water development alternatives for the Cheyenne Urban Area are summarized below.

- Increased pumpage of municipal wellfields - This alternative would have a very minor short-term effect on groundwater levels in the Cheyenne municipal wellfields. The added reduction in water level in 1984 due to a 70 acre-ft increase over the projected use of 4,000 to 7,000 acre-ft would be small and would recover as well production was curtailed in the following years when excess water supplies were available. No effect on groundwater quality would occur.

- Use of Stage II Diversion facilities - This alternative would have no effect on the groundwater system other than a potential slight increase in the recharge rate due to increased flows in Crow Creek from additional import water.

- Modifications to Crow Creek watershed system - A possible effect on groundwater from this alternative would be a potential increase in recharge if snow fences were used to decrease sublimation losses. The effect would be minor. Modification to increase the use of Brush Creek or South Crow Creek could reduce groundwater recharge

rates. The reduction in recharge could approach the increase in surface water yield, especially in dry years.

- Wastewater reuse - This alternative could have a slight effect on the groundwater system downstream of Cheyenne by decreasing wastewater effluent flows (as compared to the No Action Alternative) and subsequent infiltration to groundwater. This could result in a very minor, probably not significant improvement in groundwater quality in this area.

  In addition, it is also possible that this could result in an overall decrease in the use of groundwater by the CBPU. Because the irrigation demands which would be satisfied by the wastewater effluent occur in the peak demand season (when Cheyenne relies on its wells to provide peaking capacity), any reduction in demand allows a subsequent reduction in groundwater pumping rates. This could help to stabilize the water levels in the Cheyenne wellfields.

- Water conservation - This could also help to stabilize water levels in the Cheyenne wellfields if the program is effective at reducing peak summer water demand.

- Development of the Casper aquifer - Because this aquifer is not currently utilized in the area of potential development, any effects on groundwater would be negligible, especially considering the small amounts of water required.

- Development of the Ogallala/Arikaree aquifers - Previous study has indicated that development of this source could have a negative effect upon already reduced water levels in the area. It is possible, however, that the relatively small amounts of water required to meet project-induced demands could be withdrawn from this system with only minor short-term effects. Compliance with the procedures for acquisition of rights within the Laramie County groundwater control area would ensure that use of this source would not interfere with existing rights.

- Purchase of existing groundwater rights - Use of water from these sources would be limited to the past usage. Therefore, there would be no increase in withdrawal rates and no effect on groundwater.

Effects on Surface Water Hydrology and Quality. The effects of the various water development alternatives for the Cheyenne Urban Area are summarized below.

- Increased pumpage of municipal wellfields - The added drawdown in 1984 due to a withdrawal of an additional 70 acre-ft would have a negligible effect on surface water flows.

- Use of Stage II Diversion facilities - This alternative would have a minimal effect upon surface water flows and quality. The additional import water may lead to a slight increase in spillage from Granite Springs and Crystal Lake reservoirs and a consequent minor increase in flows in the Middle Fork of Crow Creek.

- o Modification to Crow Creek watershed - Use of snow fences to increase yield would slightly increase streamflows in Crow Creek, although most of this would be held in reservoirs or diverted. Increased use of the Brush Creek and South Crow diversions would result in decreases in streamflow in these tributaries as additional water is diverted.

- o Wastewater reuse - The effect of this alternative on the surface water system would be a minor decrease (over baseline conditions) in wastewater discharge to Crow Creek downstream of Cheyenne. This would have a marginal positive effect upon water quality in Crow Creek.

- o Water conservation - As above, the only effect would be a very slight decrease in wastewater flows.

- o Development of Casper aquifer - If this were developed at intermediate depths (1,000-3,000 feet) and at a sufficient distance from areas where the formation outcrops and flows to the surface water system, there would be no effect.

- o Development of Ogallala/Arikaree aquifers - Development of this source could potentially reduce flows in Horse Creek at high pumping rates.

- o Purchase of existing rights - This alternative would have no effect on surface water hydrology or quality.

3.5.1.2.4.2 F.E. Warren AFB

Description. Construction and operating demands at F.E. Warren AFB would be supplied by continued purchase from Cheyenne. This demand peaks at about 96 acre-ft/yr in 1986 and is 76 acre-ft/yr from 1991 onward.

Effects on Groundwater Hydrology and Quality. The CBPU will have adequate supplies to provide the required water in all years except 1984 and again in 1991 if the Stage II pipeline or other source of water is not available. The amounts of water required in these years, are small compared to the amount of groundwater pumped by the CBPU. In 1984, the CBPU will be utilizing its wellfields at an estimated rate of 4,000 to 7,000 acre-ft which is greater than the desired rate of 2,000 acre-ft/yr. The incremental amount attributable to F.E. Warren AFB would be negligible.

Effects on Surface Water Hydrology and Quality. Purchase of water from CBPU to meet project demands at F.E. Warren AFB would have no effect upon surface water hydrology or quality.

3.5.1.2.4.3 Deployment Area

Description. Construction water requirements in the DA will begin in 1984 and extend through 1988. A total of 449 acre-ft will be required and about two-thirds of this will be used in the Wyoming portion of the DA. Table 3.5-14 summarizes these requirements.

Table 3.5-14

CONSTRUCTION WATER REQUIREMENTS
IN THE DEPLOYMENT AREA
(acre-ft)

	1984	1985	1986	1987	1988	1989	1990	Total
Nebraska		0	0	56	88	5	0	149
Wyoming		8	160	123	6	3	0	300
TOTAL:		8	160	179	94	8	0	449

Alternatives to provide this construction water include:

- Use of existing Minuteman system wells - A well is located at each LCF; these wells were constructed in 1962 and at the time had discharge capacities ranging from 7.5 to 50 gpm. Maximum capacity for the wells in Wyoming averaged 23 gpm and in Nebraska averaged 16 gpm. Several wells pumped excessive quantities of sand or had extremely high drawdowns at the higher rates of production. Consequently, it is unlikely that these wells could be used for more than just a small quantity of construction water to be used near the LCF sites.

- Development of new groundwater supplies - Procedures for the appropriation of groundwater are quite different in Wyoming and Nebraska. Procedures in each state have been previously summarized in Section 2.6.2. At this time no specific aquifers have been identified as sources. This would vary throughout the DA based upon which aquifers are present under the various areas for construction. From a regulatory standpoint, the development of new groundwater sources is a more easily implemented alternative in Nebraska than in Wyoming.

- Purchase/temporary use of existing water rights - Procedures for the temporary acquisition and use of existing water rights are also summarized in Section 2.6.2. During the construction of the existing Minuteman system, water was purchased from communities throughout the area for construction use. The majority of the communities in the DA currently have excess wellfield capacities. This situation is projected to remain unchanged throughout the project-construction period. Use of this water, or purchase of water from irrigation wells throughout the area constitutes a viable alternative for providing construction water. Ertec Western, Inc. has identified existing water rights in the vicinities of all LFs, LCFs, and DARs to be upgraded in order to assess the potential for utilizing these water sources as well as identifying water supply sources within 2 miles of the municipalities in the ACS.

Effects on Groundwater Hydrology and Quality. It is not anticipated that any of the identified alternative sources of construction water would have an effect upon the quality of groundwater in the DA. Because of the short-term nature of the use and the limited quantities of water required, there would be no long-term effect upon groundwater levels. Because of regulatory requirements for the acquisition of new groundwater supplies, there should be no short-term impact on existing wells due to acquisition of new or existing water rights. Pumping rates from existing Minuteman wells are small, so they have limited impacts upon nearby wells.

Effects on Surface Water Hydrology and Quality. There would be no appreciable long-term effect upon surface water flow or quality due to the short-term, limited water use for construction. It is possible that the radius of influence of some wells, either new or existing, may interfere with a nearby stream. In selecting project water supplies, wells will be located to avoid this wherever possible. Single well equations will be used to select well spacing and pumping rate limitations.

3.5.1.2.4.4 Other Areas

Description. Project-induced demands on municipal water supplies in ACSs outside of the Cheyenne Urban Area amount to a total of 686 acre-ft over the 5-year construction period. Of this, 324 acre-ft occur in Nebraska and 362 acre-ft occur in Wyoming. For the 8 communities impacted, the induced increase in demand as a percentage of 1980 water use varies from 1.0 percent in Gering to 29 percent in Chugwater. The existing water supplies in all of these communities are adequate to meet these increased demands. There are no induced long-term increases in demand in any of these communities.

Effects on Groundwater Hydrology and Quality. There will be no appreciable impact upon groundwater levels due to increased pumpage to meet the increases in demand projected for these communities. Currently, one of Kimball's major supply wells causes some interference with a nearby irrigation well; the City is investigating the development of an additional well and operational changes that would eliminate this interference in the future. This is not anticipated to be a problem as a result of project-induced demands.

Effects on Surface Water Hydrology and Quality. There would be no impacts on the surface water system of the ROI as a result of increased use of existing municipal water supplies to meet project-induced demands.

3.5.2 Legal Requirements and Constraints

3.5.2.1 Baseline Future - No Action Alternative

No significant changes in state water laws or regulations are foreseen. If groundwater levels continue to show seasonal decline, it is possible that the groundwater control area could be expanded into the western section of Laramie County. The State Engineer's Office, however, does not expect this to happen within the next 5 years. Also, Stage II of the Cheyenne Water Diversion Project will increase the supply of water in the area, possibly reducing the demand for groundwater. No additional water rights are required to meet baseline demands in the Cheyenne Urban Area. Additional rights will be acquired in other areas primarily related to increased water use for irrigation.

3.5.2.2 Proposed Action

No significant changes in state water laws or regulations are foreseen. It is not anticipated that induced water demand will create a need for additional water rights. Groundwater development, if pursued, would be constrained by legal requirements of the States of Wyoming and Nebraska. The Air Force has committed to meeting those requirements in water development for project construction. It is expected that water needed in Wyoming will be obtained from Wyoming sources and water needed in Nebraska will be obtained from Nebraska sources to avoid potential problems in transferring water across state boundaries.

3.5.3 Surface Water Hydrology and Quality

This section describes anticipated future trends without the project (No Action Alternative) and effects or changes to these trends as a result of the Proposed Action. The discussion focuses on storm runoff and drainage, erosion and siltation, wastewater discharges, surface water quality, and surface water flow.

3.5.3.1 Baseline Future - No Action Alternative

Surface water hydrology and water quality trends without the project through 1991 cover the time period necessary for project construction and stabilization of the population to operational conditions. Surface water hydrology and water quality trends without the project during this period are controlled by growth in population and changes in land use.

Populations of the 6-county area containing the ACS watersheds are projected to increase from 137,290 in 1983 to 153,320 in 1990. Irrigated land is projected to increase from 453,000 acres to about 530,000 acres. Population increases and related development will increase the percentage of impervious area and the total surface area disturbed during construction. These changes will locally increase surface runoff volumes and peaks, reduce pervious surface area available for recharge, and increase municipal wastewater return flows. In addition, increases in surface area disturbed during construction will temporarily accelerate surface erosion.

Increased irrigated acreage within the ROI without the project will tend to degrade water quality due to evaporation-induced increases in total dissolved solids (TDS) concentrations and potential nitrogen leaching to the groundwater basin. Further increases in consumptive use in the watersheds could continue to be reflected in decreased flow of streams and creeks. Projects located near springs could also reduce spring flow.

The anticipated situation regarding the variation in surface water quality in the downstream direction will remain in effect. Water quality is generally degraded as the streams flow from west to east. This phenomenon is caused by several factors, among them the continual dissolution of minerals and salts in contact with the water, degradation due to irrigation and chemical application, losses due to evaporation, and additional impacts such as sewage disposal that are associated with the increased drainage area for each stream as a function of the distance downstream.

3.5.3.1.1 Crow Creek Watershed

Surface water hydrology and quality trends within the Crow Creek watershed through 1991 without the project are determined primarily by increased population and development within Cheyenne and increased import of transbasin water from 7,400 acre-ft/yr in 1981 to 12,400 acre-ft/yr in 1986 upon completion of the planned Stage II Water Diversion Project.

One of the effects of continuing development of the Crow Creek basin will be impacts on stormwater runoff and drainage. The conditions in 1987 are forecast to provide a basis for project impact assessment. Population growth and land use changes in the urban area have been allocated by neighborhood and township sections and further allocated to drainage basins. The disturbed area associated with this development was then calculated based on residential and other land use change (Land Use EPTR) in Cheyenne. A total of about 24 acres of partly impervious area per 1,000 people are estimated for nonresidential development of which 1.75 acres are commercial, 12 acres are industrial, and 10 acres are public and semipublic.

The total population growth over existing conditions by 1987 is projected at 4,840. The total increase in land use of an impermeable nature for the urban area by 1987 from 1983 conditions without the project is about 512 acres with 321 acres of single-family residential, 36 acres of multifamily residential, 40 acres of mobile home residential, 9 acres of commercial, 58 acres of industrial, and 48 acres public and semipublic. Allocations of the land use changes to basins and total disturbed area are shown in Table 3.5-15.

The relative change in runoff volumes can be calculated by assessing the curve number changes related to these land use changes. The TR-55 SCS method is used to assess curve number differences between developed and undeveloped watershed conditions. Table 3.5-16 shows the composite curve number by basin for existing land use and for the projected 1987 land use condition.

To estimate runoff, Q, the following relations are used:

$$Q = \frac{(P-0.2S)^2}{P+0.8S} \quad \text{for } P > 0.2S$$

$$Q = 0 \quad \text{for } P < 0.2S$$

$$S = \frac{1000}{CN} - 10$$

Where:

Q = runoff, inches;

P = precipitation, inches;

S = maximum retention, inches; and

CN = curve number.

Table 3.5-15

DEVELOPED AREA CHANGE, 1983-1987
WITHOUT THE PROJECT

Drainage Basin[1]	Land Use[2], acres SF/MH	MF	C	I	P/SP	TOTAL
C 12	-	-	-	-	-	-
C 13	-	-	-	-	-	-
C 14	-	-	-	-	-	-
C 15	7	-	1	9	-	16
C 16	-	-	1	8	2	12
C 17	1	-	1	5	7	15
C 17a						
C 18	19	-	-	2	-	19
C 19	12	-	-	3	1	17
C 20	1	-	1	13	12	28
C 20a						
C 21a	49	5	1	-	2	23
C 21b	147	18	2	1	11	94
C 21c	53	5	-	1	-	47
C 22	55	8	1	7	10	79
C 23	16	-	1	9	3	30
C 24	1	-	-	-	-	1
C 25	-	-	-	-	-	-
TOTAL:	361[a]	36	9	58	48	512
Curve Number[3]	79	88	93	90	75	-

Notes: 1 See Figure 2.6-9 for location of drainage basins.

2 SF = single family, MH = mobile homes, MF = multifamily, C = commercial, I = industrial, and P/SP = public and semi-public land uses.

3 Curve number for this type of land use.

a Includes 321 acres of single family and 40 acres of mobile homes.

Table 3.5-16

COMPOSITE CURVE NUMBER BY BASIN

Basin	Existing Curve Number	Projected Curve Number in 1987
C13	70	70
C14	70	70
C15	71.5	71.6
C16	72.2	72.3
C17	72.4	72.4
C18	71.1	71.1
C19	76.3	76.7
C20	81.6	81.8
C21a	74.1	74.3
C21b	74.6	74.9
C21c	73.9	74.0
C22	78.9	79.2
C23	72.5	72.6
C24	70	70.1
C25	70.8	70.8

Time increments of 30 minutes are used to model flows with the peak flow from the drainage basin for each interval calculated as:

$$Q_p = 484\ A\ Q/T_p$$

where:

Q_p = peak flow, cfs
A^p = drainage area in square miles
Q = runoff in inches
T_p = time to peak runoff, hours, and is defined as
$T_p = D/2 + 0.6\ T_c$, where:
D^p = duration of rainfall excess
T_c = time of concentration, hours, which is a function of basin flow length, slope, and ground cover

The hydrographs developed for each 30 minute intervals are added sequentially to determine the storm hydrograph for the basin. Rainfall is specified for the storm under consideration. A 50-year, 6-hour storm has 2.9 inches; a 10-year, 3-hour storm has 1.71 inches; and a 2-year, 24-hour storm has 1.68 inches of rainfall. The peak flows calculated by these procedures for 1987 without project conditions are shown in Table 3.5-17. The values indicate very minor changes over existing conditions for any of the basins (Table 2.6-27).

For the 2-year, 24-hour storm, one basin shows a 2 cubic foot per second (cfs) increase, and 5 basins show a 1 cfs increase. The combined flood flow increases from 1,503 cfs to 1,505 cfs, an increase of 0.13 percent. The 10-year, 3-hour storm shows similar small changes for the 1987 development without the project. The largest change is 3 cfs with 2 basins showing this change, while 1 basin shows 2 cfs and 6 other basins show a 1 cfs change. Overall, the flood flow changes from 2,096 cfs to 2,099 cfs, a 0.14-percent increase. The largest increase for the 50-year, 6-hour storm is 5 cfs in basin 21b (Dry Creek). Nine other basins show increases ranging from 1 to 4 cfs. The total flood flow increase is less than 0.1 percent from 9,068 cfs to 9,072 cfs.

This analysis is only valid for assessing the regional effects of urban development. There will be site-specific changes in stormwater flow which will have to be addressed at the time of the land use change. Changing land use has the greatest impact at the location of the change. The Utilities EPTR provides analyses of the relatively large changes that could occur as the result from development of parcels ranging in size from 20 to 160 acres and the costs of controlling this increase.

If it were assumed that the entire 512 acres of land use change were to occur on vacant land in 1 area (curve number 70), the composite new curve number for the area would be 80.8. The runoff, Q, for a 50-year, 6-hour storm would increase from 0.66 inches to 1.22 inches. Peak flow would increase from 98 cfs to 283 cfs. Peak flows from smaller areas would be proportionate to this increase. Structured development following the standards and criteria established by the City of Cheyenne should be pursued so as not to drastically increase the operation and maintenance costs of the existing stormwater system.

Table 3.5-17

FLOOD FLOWS IN CHEYENNE IN 1987 WITHOUT THE PROJECT

Basin	Name	Area, Mi^2	CN	2-Year - 24-Hour		10-Year - 3-Hour		50-Year - 6-Hour	
				TP(hr)	Peak Flow(cfs)	TP(hr)	Peak Flow(cfs)	TP(hr)	Peak Flow
Crow Creek @ Diamond		248		34	1,279	18	1,815	20	7,741
C12	Diamond	10.89		30	37	19	42	21	194
C13	Cigar Creek	4.25	70	19	27	8	42	10	189
C14	Clear	9.51	70	29	33	19	38	20	173
C15	Clear	10.28	71.6	22	63	12	80	13	333
C16	West Cheyenne	7.57	72.3	19	61	8	95	10	372
C17	West Cheyenne	10.83	72.4	25	64	14	78	16	312
C17a	Cheyenne	1.20		1.17	24	1	300	1	300
C18	South Cheyenne	9.43	71.1	18	66	8	107	9	451
C19	South Cheyenne	1.29	76.7	16	27	6	59	7	184
C20	East Cheyenne	.74	81.8	16	15	6	34	7	88
C20a	East Cheyenne	1.27		1	120	1	120	1	120
C21a	Dry Creek	2.97	74.3	18	30	7	55	9	192
C21b	Dry Creek	3.07	74.9	19	53	8	87	10	301
C21c	Dry Creek	5.91	74.0	18	58	8	98	9	352
C22	East Cheyenne	4.76	79.2	22	61	11	84	13	250
C23	South Cheyenne	9.58	72.6	19	80	9	121	10	469
C24	Kingman Ditch	7.66	70.1	22	40	12	51	13	230
C25	Bolla Ditch	5.82	70.8	18	42	8	66	9.45	280
Crow Creek @ Hereford Ranch Reservoir		346		40	1,505	23	2,099	26	9,072

Erosion and siltation in the Crow Creek watershed will increase due to increased population and associated construction. Population increases projected for Cheyenne without the project average 1,410 persons per year between 1985 and 1990. Construction related to this increase would disturb an average of 151 acres per year based on residential land use changes and an assumed 24 acres per 1,000 persons of other urban development, which would increase erosion. Estimated erosion from construction disturbed land is 75 tons per acre (T/acre), resulting in approximately 11,300 tons per year (T/yr). For the portion of Crow Creek between Cheyenne and Carpenter, this would increase existing erosion from 481,400 tons to 492,700 tons in 1985, or 2.4 percent. For other segments of Crow Creek, erosion rates are expected to remain unchanged from 1983 levels, as little or no change in population is predicted.

Surface water quality in Crow Creek is dependent on the efficiency and level of wastewater treatment provided by the three sewage treatment plants in the Cheyenne area. Without the project the quality of the effluent and therefore the quality of the water in Crow Creek will continue to decline as the population grows and the sewage facilities continue to be overloaded.

In 1980 the effluent from the three facilities produced a combined BOD_5 load on Crow Creek of approximately 1,600 pounds per day (lb/day) and 1,350 lb/day of total suspended solids (TSS). Two facilities, Cheyenne's Crow Creek and Dry Creek wastewater treatment plants, are marginally meeting their effluent requirements in some months of the year but on an annual average achieve better than secondary levels. The South Cheyenne treatment plant on an annual basis is not meeting the requirement of 30 milligrams per liter (mg/l) BOD and 30 mg/l TSS. In the future, these waste treatment facilities will require some type of up-grade and/or enlargement (Banner Associates 1982). The status of these plans is discussed more fully in the Utilities EPTR. Until the sewage facilities are upgraded the water quality in the Crow Creek drainage will continue to decline. If treatment levels and removal efficiencies remain at present levels, the predicted loads on Crow Creek are shown in Table 3.5-18. The assumption that efficiencies will remain constant is not entirely valid, because as the treatment facilities are being overloaded more each year, it is likely the treatment efficiency will decline. However, identifying and quantifying this decrease in efficiency is difficult. It could range from a negligible decrease to up to 30 percent drop in efficiency. Wastewater effluent would represent 92 percent of the flow in Crow Creek downstream of Cheyenne without the project.

3.5.3.1.2 Lodgepole Creek Watershed

Surface water hydrology and water quality trends are expected to parallel populations and land use projections with most impacts related to changes in land use. No significant population growth is expected in the watershed without the project. Additional agricultural lands are expected to be placed into cultivation with increased irrigation on some lands.

No major change in impervious surface is expected in the watershed so no significant change in flooding conditions would be expected.

Changes in erosion and siltation effects would not be expected to be significantly altered from baseline conditions since no significant population

Table 3.5-18

WASTEWATER LOADS ON CROW CREEK FROM CHEYENNE URBAN AREA

Baseline Year	1983	1984	1985	1986	1987	1988	1989	1990	1991
Population Served	58,954	59,565	60,869	61,924	63,182	64,421	65,772	67,048	68,436
Sewage Flow (mgd)[1]	8.58	8.66	8.85	9.01	9.19	9.38	9.56	9.75	9.96
Load to Crow Creek if Secondary Treatment Quality is Achieved[2]									
BOD_5 (tons/yr)	391	395	404	411	419	428	436	445	454
TSS (tons/yr)	391	395	404	411	419	428	436	445	454

Notes: 1 Based on forecasts in Utilities EPTR.

2 Secondary efficiency is 30 mg/l BOD_5 (biochemical oxygen demand) and 30 mg/l TSS (total suspended solids). The table indicates historic treatment efficiency is better than secondary treatment quality.

changes are expected. Construction projects within the basin could increase erosion and sediment delivery; however, no planned large projects of this nature are known at this time.

Increased groundwater pumping could reduce surface water flow and flow from springs in the watershed, but is not expected to significantly change water quality. Groundwater pumping upstream of Bushnell, Nebraska has reduced annual flows there from about 13 cfs to about 5 cfs. Because most of this area is in the Laramie County groundwater control area which restricts new wells, a similar decrease in flow is not expected in the future.

Lodgepole Creek presently receives discharge from two population centers, Kimball and Bushnell. All other cities and towns in the ROI utilize zero discharge systems or are not served by sewers. Bushnell may convert to a zero discharge facility in the future; however, no planning is available to determine when and how this will be done.

Presently Bushnell discharges 0.02 million gallons per day (mgd) of sewage treated in a lagoon with an effluent quality of 30 mg/l BOD_5 and 80 mg/l TSS resulting in a load to Lodgepole Creek of 0.91 T/yr BOD_5 and 2.44 T/yr TSS. This is not expected to change by a significant amount over the 1983 to 1990 period.

Kimball utilizes an oxidation ditch treatment system that discharges 0.66 mgd of secondary effluent resulting in a loading to Lodgepole Creek of 30.1 T/yr BOD_5 and TSS. Because the baseline population is expected to decrease by 1.8 percent, the load to the creek will be reduced by a similar amount resulting in a 1990 loading of 0.65 mgd and 29.6 T/yr of both BOD_5 and TSS.

3.5.3.1.3 Horse Creek Watershed

It is anticipated that the trends in hydrology and water quality will follow the same pattern as those in Lodgepole Creek. No major population changes are expected in the watershed and most future impacts will be related to changes in the use of land for agricultural purposes.

No municipality in the watershed has caused surface water quality problems, and it is unlikely that problems will arise in the future.

3.5.3.1.4 Pumpkin Creek Watershed

The Pumpkin Creek drainage has very little developed land and is principally an agricultural area. Irrigation is expected to increase with the major portion of the increase to be obtained from groundwater. Since most of the increase in irrigated land will come from groundwater, there is little chance that surface water will have significant water quality changes. This is because most of the applied groundwater is consumptively used and produces little or no return flow. Groundwater use may affect water quality by lowering streamflows. However, this is not expected to be a large change. Pumping of certain aquifers may cause seeps and springs that once flowed to the surface water to dry up. This phenomenon may remove some chemical inputs to the surface water, thus decreasing concentrations of certain chemical parameters. The chemical composition of the water may be altered slightly but the overall concentration of dissolved matter would be approximately the same as existing conditions.

3.5.3.1.5 Chugwater Creek Watershed

This watershed will show changes similar to those in Lodgepole and Horse creeks. Agricultural practices will continue to be the most important factor influencing the watershed. Wastewater flow from Chugwater is approximately 0.2 mgd but does not directly flow to the creek.

Construction within the watershed could foreseeably cause erosion and sedimentation and altered flow patterns; however, no information indicates that large construction projects are planned.

3.5.3.1.6 Other Watersheds

Other watersheds in the ROI would experience changes similar to the preceding watersheds:

- o Decreased flow where increasing irrigation reduces groundwater recharge;
- o Decreased quality where wastewaters or irrigation return flows would change from existing conditions;
- o Increased flooding where the degree of imperviousness of the watershed changes; and
- o Increased erosion where construction activities are not managed to control erosion.

These changes are not easily quantified.

3.5.3.2 Proposed Action

3.5.3.2.1 Storm Runoff and Drainage

A major factor affecting surface runoff is tributary drainage area. Total drainage area tributary to the main stream of the watersheds in the ROI increases in the downstream direction to the extent that small changes in the upper watershed have little effect on surface runoff and drainage at the downstream boundaries of the watersheds.

The key factor influencing surface runoff that would change as a result of the Proposed Action is the percentage of impervious area. Increased impervious area within a basin tends to increase surface runoff volumes and peak flow rates by increasing the amount of direct surface runoff and by decreasing basin lag times by creating smoother, more efficient flow patterns.

The Crow Creek watershed will be impacted mainly in the Cheyenne Urban Area due to increases in population and subsequent land use changes. Other watersheds will have more direct, localized impacts due to construction and operating activities at the LF sites, DAR roads, and cable paths.

3.5.3.2.1.1 Crow Creek Watershed

The Crow Creek watershed will have direct impacts at F.E. Warren AFB and indirect impacts that will occur in the Cheyenne Urban Area. Approximately 50 acres of land will change from pervious to impervious surface at F.E. Warren AFB. This will occur over a large area and will not materially change flow from the base to Crow Creek in basins C-16 and C-17. Based on an overall curve number change from 70 to 98, the peak flow increase could be from 22 cfs to 58 cfs from the 50 acres of new development on F.E. Warren AFB due to a 50-year, 6-hour storm. This would be attenuated in overland and channel flow and would likely cause no measureable change in flood flows in Crow Creek itself.

Total inmigration to the Cheyenne Urban Area in 1987 will reach 2,640 people. These people will create a net demand for 33 acres of single-family residential, 24 acres of mobile home residential, and 1 acre of multifamily residential. To be conservative, the developed acres of industrial, commercial, and public land uses per 1,000 population increase is also applied. This results in 5 acres commercial, 32 acres industrial, and 26 acres of public and semipublic land use change. Allocation of these land use changes to the drainage basins is shown in Table 3.5-19.

Table 3.5-20 shows the composite curve number for the projected 1987 land use without the project and the projected land use in that year with the project.

Using the procedures of Section 3.5.3.1.1, Table 3.5-21 shows the peak flows generated from the individual sub-basins of the Cheyenne Urban Area. Table 3.5-22 summarizes the peak flows for each of the three land use conditions for the three separate storms. What is particularly notable about the peak flows is not just the extremely small percentage increase from existing conditions to 1987 baseline conditions (mentioned in Section 3.5.3.1.1), but the almost imperceptible increase in flows from the 1987 baseline conditions to the 1987 impact conditions.

Slight changes are visible particularly in the Dry Creek areas and also in the areas of east Cheyenne. The total increase in flow at Hereford Ranch Reservoir for each of the storms is extremely small. The 2-year, 24-hour storm shows an increase of 0.2 percent from the 1987 baseline to 1987 impacts condition. The 10-year, 3-hour storm and the 50-year, 6-hour storm show changes of 0.1 percent and 0.05 percent, respectively, between 1987 baseline and project impact scenarios.

With continuing development, the peak flow for each basin increases, while the time-to-peak decreases. Although this is not shown directly in Table 3.5-22, it is evident by the increases in peak flows at Hereford Ranch Reservoir.

The total peak flow from the 122 acres of developed area could increase from 39 to 114 cfs for a 50-year, 6-hour storm and this level of increase could cause local flooding problems. The basin analysis indicates the peak flow will be attenuated by overland and channel flow and only cause slight Creek from this entire area would not be altered greatly by increases in land use through 1987, but local nuisance conditions could be further compounded if steps were not taken to mitigate the increased flow from local development.

Table 3.5-19

DEVELOPED AREA CHANGE, 1983-1987
DUE TO THE PROJECT

Drainage Basin[1]	Land Use[2], acres SF/MH	MF	C	I	P/SP	TOTAL
C 12						
C 13						
C 14						
C 15	1	-	-	5	-	6
C 16	-	-	1	5	1	7
C 17	-	-	-	3	4	7
C 17a						
C 18	8	-	-	2	-	10
C 19	2	-	-	2	1	5
C 20	-	-	1	4	6	11
C 20a						
C 21a	10	-	-	-	1	11
C 21b	18	-	1	1	6	26
C 21c	6	2	-	1	-	9
C 22	6	-	1	3	5	15
C 23	6	-	1	5	2	14
C 24	-	-	-	1	-	1
C 25	-	-	-	-	-	-
TOTAL:	57[a]	2	5	32	26	122
Curve Number[3]	79	88	93	90	75	-

Notes: 1 See Figure 2.6-9 for location of drainage basins.

2 SF = single family, MH = mobile homes, MF = multifamily, C = commercial, I = industrial, and P/SP = public and semi-public land uses.

3 Curve number for this type of land use.

a Includes 83 acres of single family and 24 acres of mobile homes.

Table 3.5-20

COMPOSITE CURVE NUMBERS BASED ON LAND USE

Basin	1987 W/O Project	1987 W/Project
C13	70	70
C14	70	70
C15	71.6	71.6
C16	72.3	72.3
C17	72.4	72.4
C18	71.1	71.1
C19	76.7	76.7
C20	81.8	82.0
C21a	74.3	74.4
C21b	74.9	75.1
C21c	74.0	74.1
C22	79.2	79.4
C23	72.6	72.6
C24	70.1	70.1
C25	70.8	70.8

Table 3.5-21

FLOOD FLOWS IN 1987 IN CHEYENNE WITH THE PROJECT

Basin	Name	Area, Mi^2	CN	2-Year - 24-Hour		10-Year - 3-Hour		50-Year - 6-Hour	
				TP(hr)	Peak Flow(cfs)	TP(hr)	Peak Flow(cfs)	TP(hr)	Peak Flow
Crow Creek @ Diamond		248		34	1,279	18	1,815	20	7,741
C12	Diamond	10.89	70	30	37	19	42	21	194
C13	Clear Creek	4.25	70	19	27	8	42	10	189
C14	Clear Creek	9.51	70	29	33	19	38	20	173
C15	Clear Creek	10.28	71.6	22	63	12	80	13	333
C16	West Cheyenne	7.57	72.3	19	61	8	95	10	373
C17	West Cheyenne	10.83	72.4	25	64	14	78	16	313
C17a	Cheyenne	1.20		1	24	1	300	1	300
C18	South Cheyenne	9.43	71.1	18	66	8	108	9	451
C19	South Cheyenne	1.29	76.7	16	27	6	60	7	185
C20	East Cheyenne	.74	82.0	16	15	6	35	7	88
C20a	East Cheyenne	1.27		1	120	1	120	1	
C21a	Dry Creek	2.97	74.4	18	31	7	56	9	194
C21b	Dry Creek	3.07	75.1	19	54	8	89	10	305
C21c	Dry Creek	5.91	74.1	18	59	8	99	9	353
C22	East Cheyenne	4.76	79.4	22	62	11	85	13	252
C23	South Cheyenne	9.58	72.6	19	80	9	121	10	469
C24	Kingman Ditch	7.66	70.1	22	40	12	51	13	230
C25	Bolla Ditch	5.82	70.8	18	42	7.95	66	9	280
Crow Creek @ Hereford Ranch Reservoir		346		40	1,506	23	2,100	26	9,074

Table 3.5-22

SUMMARY OF FLOOD FLOWS IN CHEYENNE

Basin	Name	2-Year - 24-Hour			10-Year - 3-Hour			50-Year - 6-Hour		
		Existing	Baseline 87	Impact	Existing	Baseline 87	Impact	Existing	Baseline 87	Impact
Crow Creek @ Diamond Creek		1,279	1,279	1,279	1,815	1,815	1,815	7,741	7,741	7,741
C12	Diamond Creek	37	37	37	42	42	42	194	194	194
C13	Clear Creek	27	27	27	42	42	42	189	189	189
C14	Clear Creek	33	33	33	38	38	38	173	173	173
C15	Clear Creek	62	63	63	79	80	80	332	333	333
C16	West Cheyenne	61	61	61	94	95	95	371	372	373
C17	West Cheyenne	64	64	64	78	78	78	312	312	313
C17a	Cheyenne	24	24	24	300	300	300	300	300	300
C18	South Cheyenne	66	66	66	108	107	108	451	451	451
C19	South Cheyenne	26	27	27	58	59	60	181	184	185
C20	East Cheyenne	15	15	15	34	34	35	87	88	88
C20a	East Cheyenne	120	120	120	120	120	120	120	120	120
C21a	Dry Creek	30	30	31	54	55	56	190	192	194
C21b	Dry Creek	52	53	54	84	87	89	296	301	305
C21c	Dry Creek	58	58	59	97	98	99	350	352	353
C22	East Cheyenne	1	61	62	81	84	85	246	250	252
C23	South Cheyenne	79	80	80	119	121	121	466	469	469
C24	Kingman Ditch	39	40	40	50	51	51	228	230	230
C25	Bolla Ditch	42	42	42	66	66	66	280	280	280
Crow Creek @ Hereford Ranch Reservoir		1,503	1,505	1,506	2,096	2,099	2,100	9,068	9,072	9,074

3.5.3.2.1.2 Other Watersheds

Other watersheds will be directly impacted by the construction located at LFs and LCFs. The impacts at individual sites on local drainage and stormwater runoff will be directly proportional to the amount of additional impervious area occurring. The assumption is made that 0.05 acres of additional impervious area per LF will result due to the project. The area of site impact includes a 1,000-foot radius around the LF. A local area of impact includes a radius of 2 miles around the LF. Regional impact is on a basin area basis.

For an average site, the total existing impervious area is approximately 1 acre. This impervious area is given a curve number value of 98. All other area is assigned a value of 70. For the site impact assessment, a 1,000-foot radius is equal to 72.12 acres of area. A composite curve number calculation for the area as it currently exists is shown below.

Area	x	Curve Number	=	Subtotals
71.12 acres		70		4,978
1 acre		98		98
72.12		70.4		5,076

The additional 0.05 acre of impervious area due to the project will change the curve number from 70.38 to 70.41, indicating a potential increase of 0.04 percent runoff in the site area. On a local basis, the total area is 8,042 acres. The composite curve number is 70.0. There is no calculable change in curve number, on a local basis, due to proposed project impacts. The local impact, based on curve number, is negligible.

On a regional basis, the cumulative impact of several sites within one basin must be assessed. Table 3.5-23 shows the total number of LFs in the other watersheds.

Lodgepole Creek Basin, with its 33 sites, will experience an increased impervious area of 1.7 acres due to final construction of the project. Detailed calculations are not necessary to predict that an additional 1.7 acres of impervious area will not significantly affect regional drainage. Local drainage may be impacted slightly, but not substantially.

New highway construction will represent a very minor change in imperviousness. Some resurfacing of existing roads will occur but would not have an effect on current drainage systems.

Table 3.5-23

IMPERVIOUS AREA AND LAUNCH FACILITIES BY WATERSHED

Basin	Total Number of LFs	Increased Impervious Area (Acres)
Lodgepole	33	1.7
Pumpkin	17	0.9
Horse	30	1.5
Chugwater	9	0.5
North Platte	11	0.6

The other basins will also experience the same level of impact which, again, will not be large.

3.5.3.2.2 Erosion and Siltation

Project impacts are related to soil disturbance for construction activities. For construction-disturbed land, an erosion rate of 48,000 tons per square mile or 75 T/acre, is estimated (EPA 1982), if mitigation is not applied. This is the most conservative, worst-case, approach; however, with the mitigation assumed in Section 3.2.2 as part of standard Air Force construction practice, the actual amount of erosion will be much less. The Geologic Resources EPTR estimates successful application of mitigation measures would limit total erosion to 1 to 1.6 T/acre. Disturbance areas are based on housing and infrastructure needs due to population increases, and for the project due to roads, cable paths, and LFs.

3.5.3.2.2.1 Crow Creek Watershed

Project impacts on Crow Creek erosion and siltation are due to induced construction in Cheyenne, road upgrades in the watershed, and project-related construction disturbance of 132 acres on F.E. Warren AFB. No LFs, LCFs, or cable routes are located in the Crow Creek watershed.

For the Crow Creek basin upstream of Cheyenne, the only impact on erosion is due to F.E. Warren AFB construction. For 132 acres, the estimated increase in erosion of 9,900 tons, without mitigation, would increase watershed erosion 1.4 percent, to 738,800 tons, assuming all land disturbance takes place in a single year. Under this worst-case assumption, sediment delivery would also be expected to increase 1.4 percent, to 120,000 tons.

Between Cheyenne and Carpenter, road upgrades and population-induced construction in and around Cheyenne would increase estimated erosion. The peak land use impact due to population increases is 59.3 acres in 1986, of which 37.33 acres are related to housing and the remainder to commercial, industrial, and public facilities development. Project-induced population growth in that year is 925 persons. An additional 34.7 acres would be disturbed due to road upgrading, for a total of 93.9 acres.

The resulting erosion of 7,000 tons represents 1.4 percent of the predicted 492,000 tons under the No Action Alternative. Sediment delivery is also estimated to increase 1.4 percent. Table 3.5-24 summarizes the project impacts.

Alternative Routing of Roads on F.E. Warren AFB. The three possible routes for roads at F.E. Warren AFB have a slight effect on the erosion and siltation effects within Crow Creek. These alternatives are shown on Figures 3.5-2, 3.5-3, and 3.5-4. This analysis compares these three alternatives. The disturbed area due to roadway construction or upgrade is included in the 132 acres of assumed disturbed area on F.E. Warren AFB. The length of roadway and number of stream crossings are important variables in the determination of erosion.

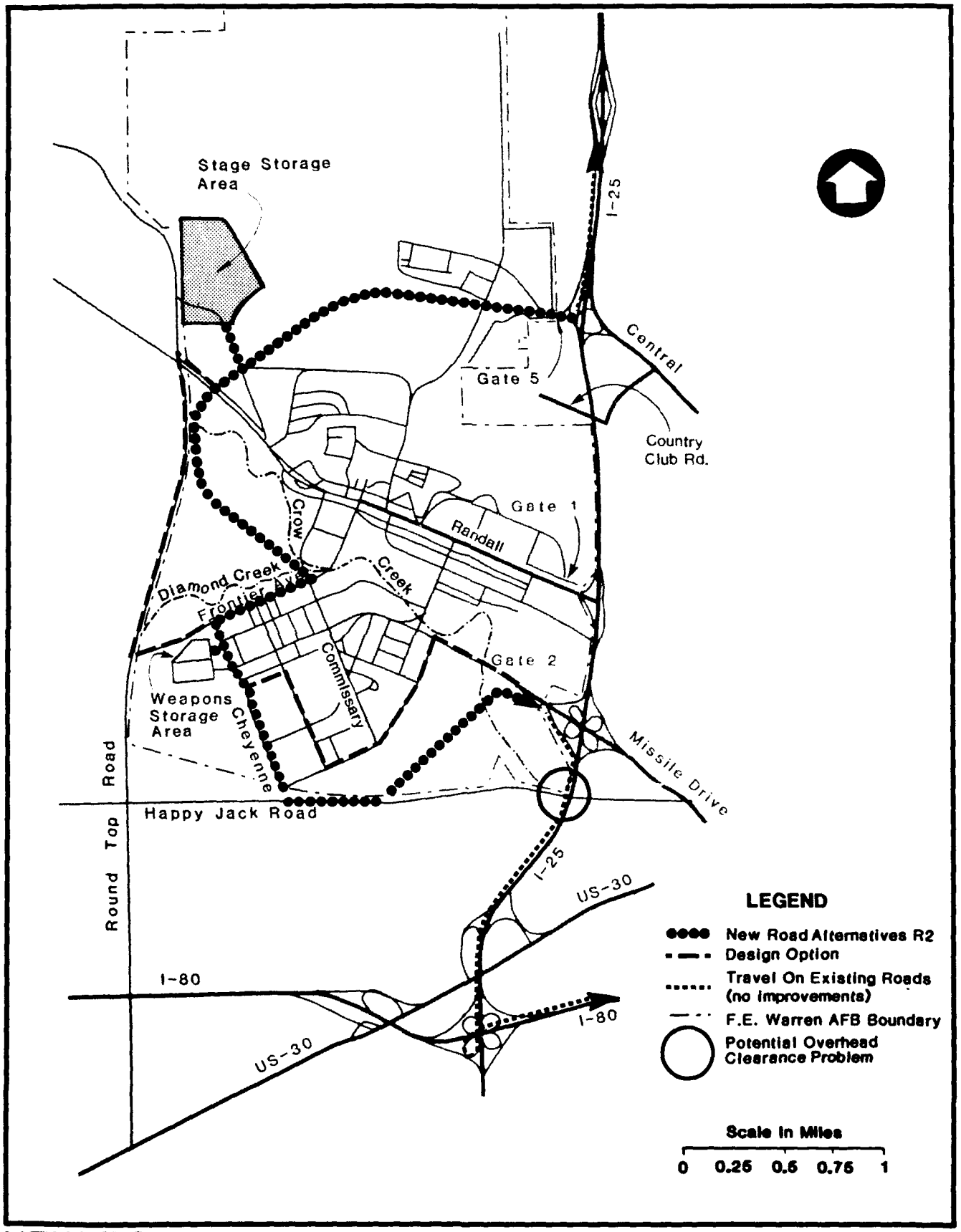
Table 3.5-25 indicates length of roadway, tons of sediment generated, and number of stream crossings. (Streams traversed include Crow Creek and Diamond Creek.)

It appears from Table 3.5-25 that Alternatives R1 and R3 are more acceptable from an erosion standpoint due to lower sediment delivery and fewer stream crossings. Selecting an alternative from R1 and R3 based on this analysis is not recommended because of the "order of magnitude" type assumptions used. Since the area in question is very small in comparison to the area on which the erosion and siltation assumptions were based and the difference between all alternatives is slight, it is difficult to assess the differences between erosion and sediment delivery between the alternatives.

Table 3.5-24

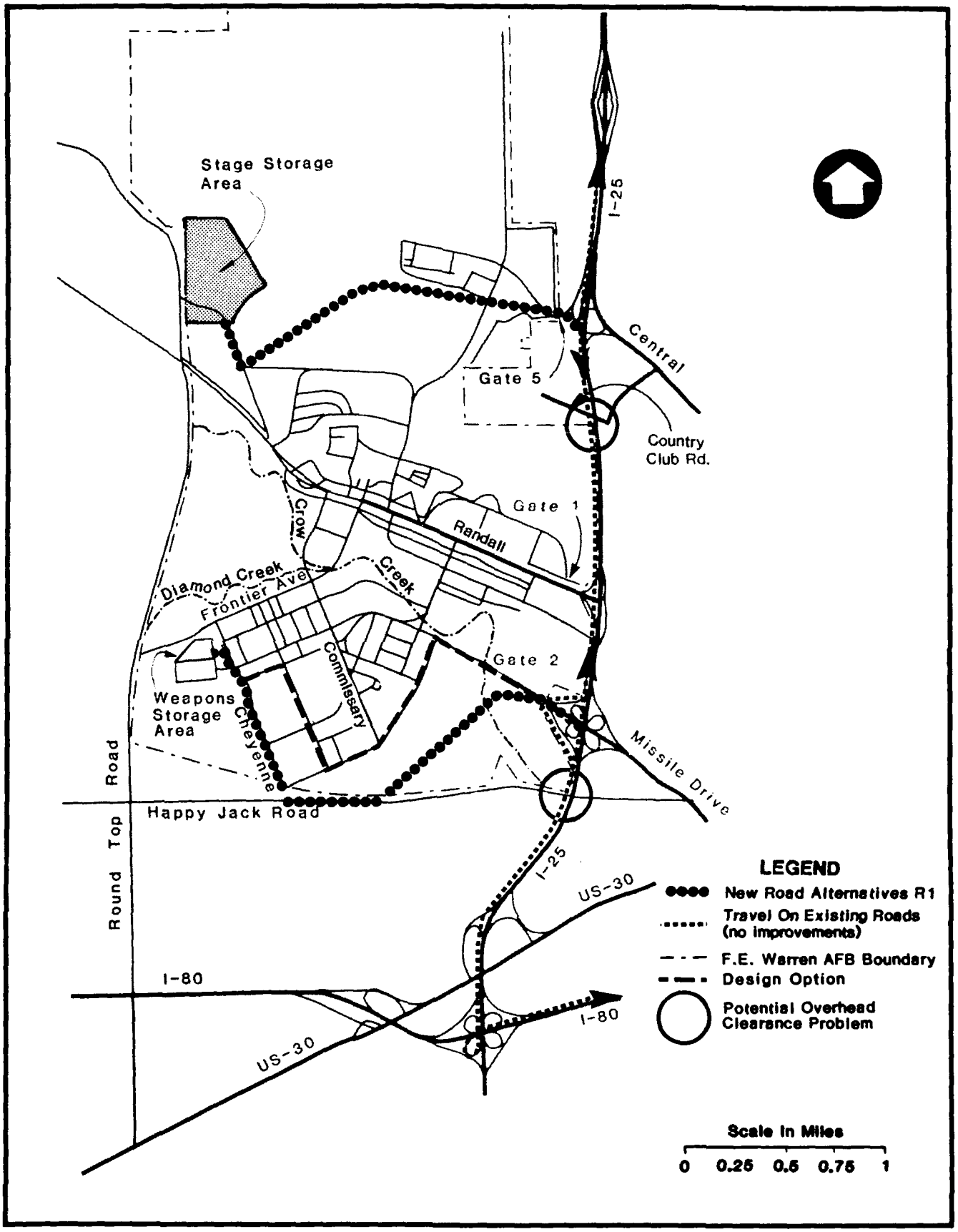
CROW CREEK EROSION AND SEDIMENTATION DUE TO PROPOSED ACTION

	Acres Disturbed	Tons of Erosion	Tons of Sediment Delivery	Increase Over No Action Percent
Above Cheyenne	132	9,900	1,600	1.4
Between Cheyenne and Carpenter	94	7,000	1,300	1.4



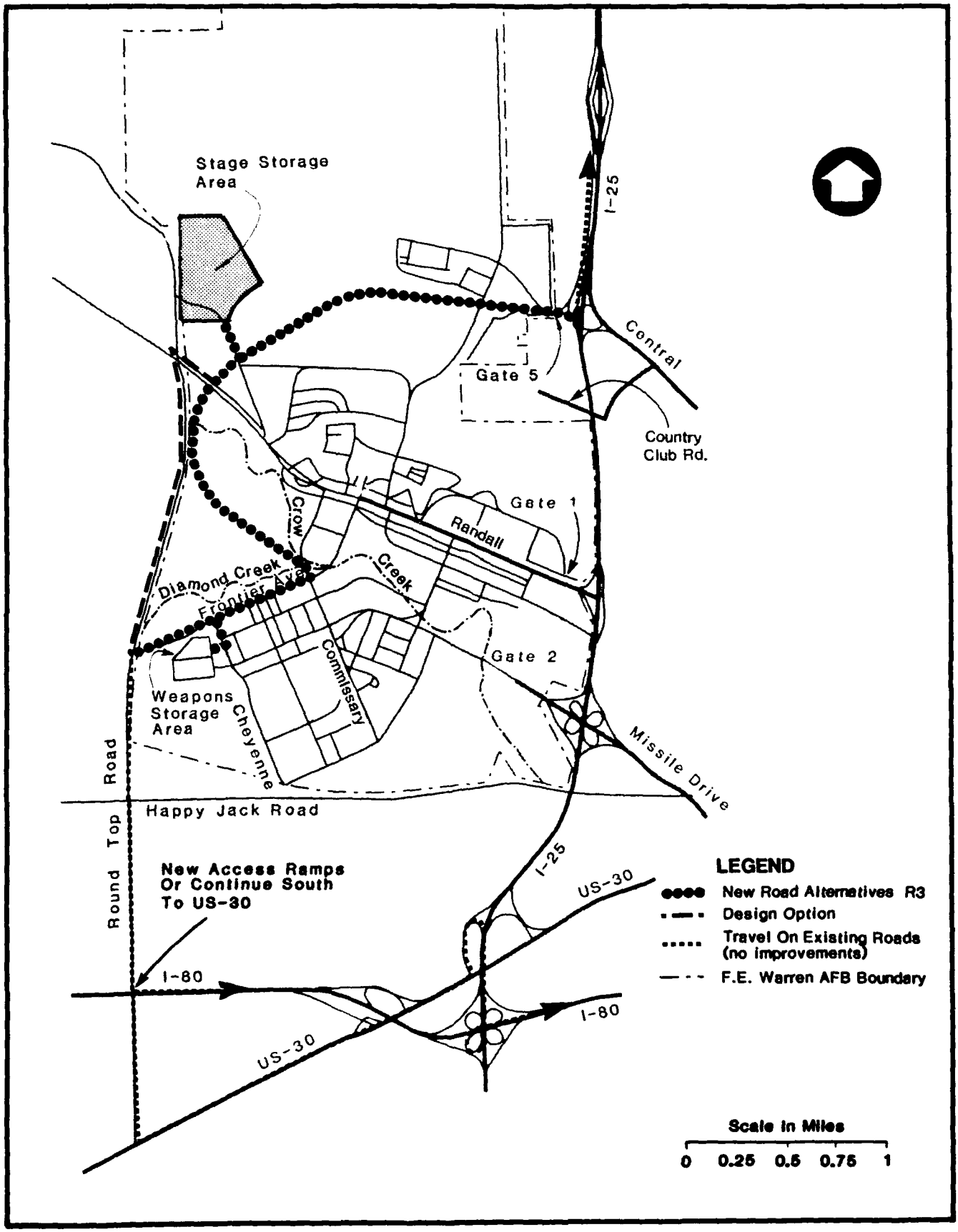
NEW ROADS AT F.E. WARREN AFB: PROPOSED ACTION R2

FIGURE NO. 3.5-2



NEW ROADS AT F.E. WARREN AFB: ALTERNATIVE R1

FIGURE NO. 3.5-3



NEW ROADS AT F.E. WARREN AFB:
ALTERNATIVE: R3

FIGURE NO.
3.5-4

Table 3.5-25

ALTERNATE F.E. WARREN AFB ROAD ROUTING - EROSION AND SEDIMENTATION

Route	Length of Road	Acres Disturbed	Tons of Erosion	Tons of Sediment Delivered	Stream Crossings
R1	5.25	47	3,500	580	1
R2	7.25	65	4,900	800	3
R3	6.25	56	4,200	690	2

No other water resource consequences are expected due to the selection of a different route for the roads on F.E. Warren AFB.

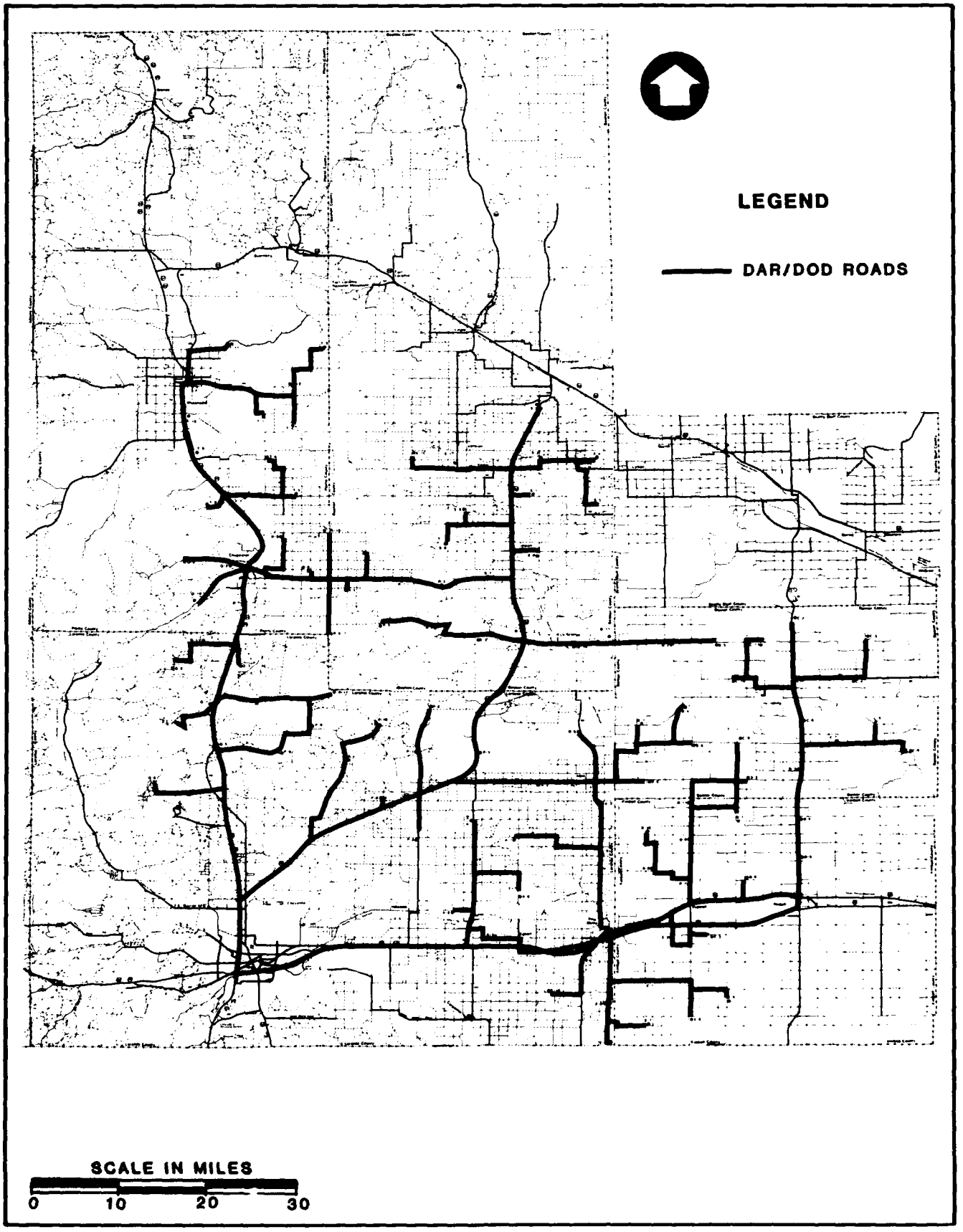
3.5.3.2.2.2 Other Watersheds

In watersheds other than Crow Creek, erosion potential would increase due to population-induced construction and construction directly associated with the project. Project construction includes roads shown on Figure 3.5-5, cable routes, and onsite construction at LFs. Population increases are assumed to be housed primarily in existing vacant housing with little net land use change and is therefore not assessed. Table 3.5-26 summarizes the areas disturbed for each watershed.

Based on the assumption of 75 T/acre without mitigation, the erosion impact due to the Proposed Action is seen to vary from 0.0 percent (Lodgepole Creek below Sidney Draw) to 1.4 percent (Horse Creek below Hawks Springs Reservoir). Sediment delivery would increase over baseline conditions without the project by the same percentage.

Erosion impacts at the LF site can be estimated by considering a site area. Based on typical distances to drainage channels from LFs, a radius of 1,000 feet is used to define the site. Impact at the site will involve the disturbance of about 0.5 acres during construction. Current usage is assumed to be cropland in order to minimize baseline erosion and provide worst-case impact. In that case, baseline erosion is estimated to be 220 tons, and disturbance would add 36 tons to that, for an increase of 16 percent. Local erosion impacts within a 2-mile radius are estimated to add 0.15 percent to the baseline erosion of 25,000 tons, using the same method. However, with mitigation, the actual values will be much less. For road upgrade, the site erosion within 1,000 feet could increase 25 percent and the local area by 2.3 percent again, indicating a need for erosion control measures.

The impacts related to the cable system are those associated with erosion and siltation. The factors controlling erosion and siltation are disturbed area and number of stream crossings. Crossing of perennial streams would have increased erosion over that encountered on dry land. Ephemeral stream crossings would be accomplished when no water is flowing; thus, erosion impacts would be similar to those on dry ground. It is assumed that for cable installation, ground disturbance is 2.4 acres per mile of cable and erosion from disturbed ground is 75 T/acre. Based on these assumptions, the erosion



TRANSPORTATION SYSTEM
SHOWING DAR

FIGURE NO. 3.5-5

Table 3.5-26

CONSTRUCTION SOIL DISTURBANCE IN WATERSHEDS DUE TO PROPOSED ACTION

Basin	Total Project-Related Construction (Acres)	Erosion due to Project Construction (Tons)[1]	Increase Over No Action Alternative (Percent)
Lodgepole Creek			
Above Oliver Reservoir	262	19,600	0.8
Oliver Reservoir to Sidney Draw	204	15,300	1.2
Sidney Draw	34	2,600	0.2
Sidney Draw to South Platte	-	-	0
Horse Creek			
Above Hawks Reservoir	320	24,000	1.4
Hawks Reservoir to North Platte	155	11,600	0.5
Pumpkin Creek to North Platte	123	9.200	0.4
Chugwater Creek to North Platte	116	8.700	0.5
Drainage to North Platte River			
Laramie River to Horse Creek	9	1,400	0.1
Horse Creek to Pumpkin Creek	-	-	0

Note: 1 Prior to assumed mitigations

and siltation impacts for cables are proportional to the length of installed cable. The possible routes for the ten cables are shown on Figure 3.5-6. Table 3.5-27 shows each alternative length, potential erosion, and number of stream crossings.

Site erosion impacts for cables can be estimated by considering a distance of 1,000 feet to either side of the cable route. Assuming that current usage is cropland, a worst-case impact of 23 percent is calculated. Local impacts within 2 miles of the cable are similarly estimated to yield 2.2 percent.

Table 3.5-27

EROSION FROM CABLE ALTERNATIVES

	Proposed Action					Alternatives					
	PA1	SB1	RB1	PA4	PA5	SB2	PB1	PD1	PA2	PA3	RB2
Length (Mi.)	1	12	25	22	24	26	20	28	13	14	14
Erosion (Tons)	180	2,200	4,500	4,000	4,400	4,700	3,600	5,100	2,400	2,500	2,500
Stream Crossing (#)	0	2	4	5	4	5	4	3	0	3	3

The Proposed Action adds about 15,300 T/yr of erosion. The lowest combination of alternatives that still supplies 5 cable interconnections would cause 9,800 T/yr (PA1, SB1, PA2, PA3, RB2).

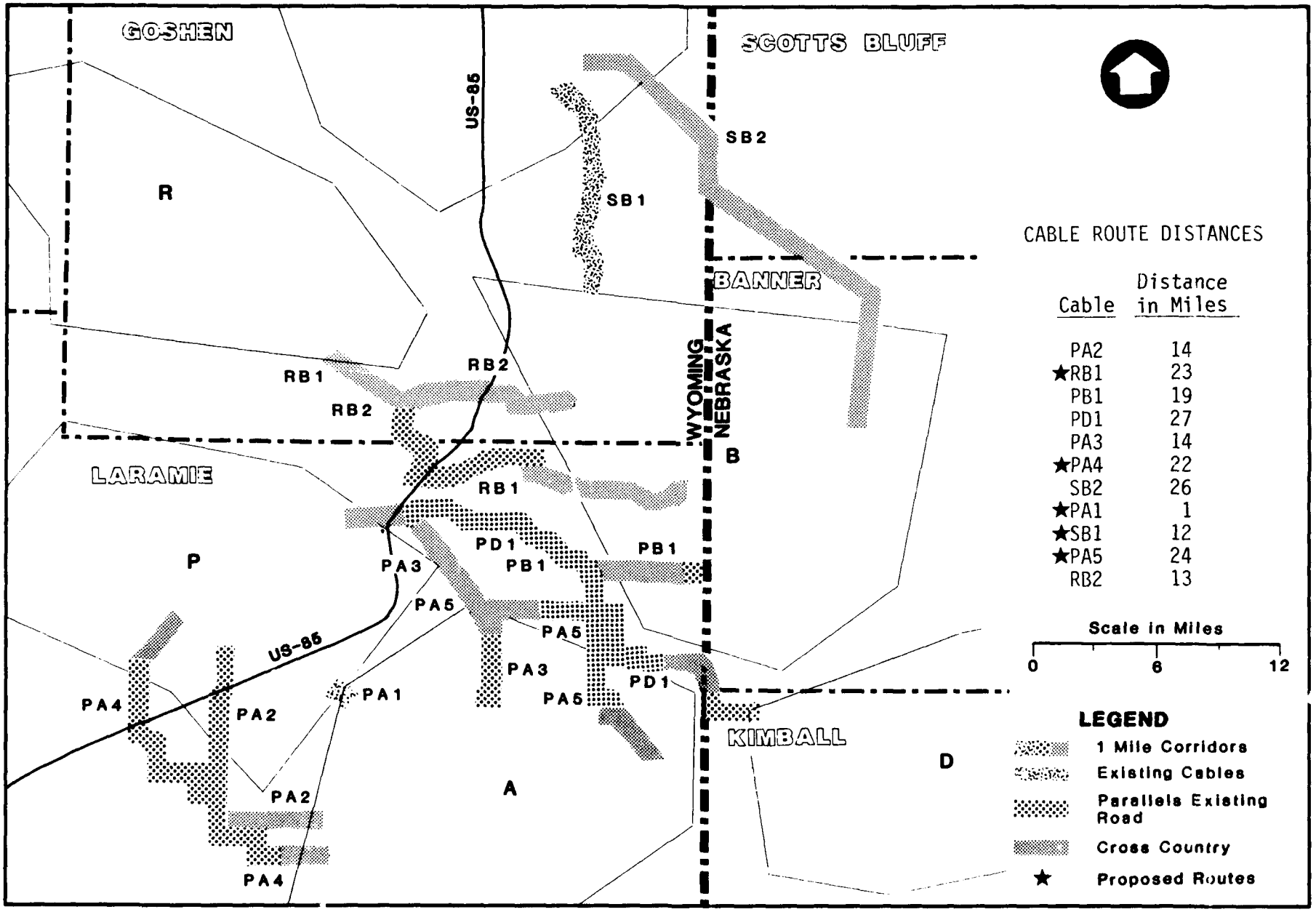
The Proposed Action would entail 15 stream crossings, while the minimum number of crossings could be 8 by using PA1, SB1, PA2, PA3, and RB2. No other areas would be impacted by selection of alternate cable routes.

3.5.3.2.3 Wastewater Discharges

3.5.3.2.3.1 Crow Creek Watershed

The three sewage treatment facilities in the Cheyenne area will experience an increase in flows due to the influx of project-related personnel. These people will cause higher mass loadings of pollutants on Crow Creek. Table 3.5-28 shows the anticipated loads on Crow Creek based on secondary treatment efficiency. Also included are the project-induced portions of each component. The peak year for the effects of the Proposed Action is 1987 when the induced portion of the load constitutes 4.0 percent of the total flow, BOD_5, TSS, and ammonia.

The typical flow in Crow Creek downstream of Cheyenne is presently composed of 92 percent treated Cheyenne area effluents and 8 percent natural flow. During the peak year this fraction will be 92.4 percent, an increase of less than one-half of 1 percent.



ALTERNATIVE CABLE ROUTES

FIGURE NO. 3.5-6

Table 3.5-28

PROJECT EFFECTS ON WASTEWATER LOADS TO CROW CREEK

	1983	1984	1985	1986	1987	1988	1989	1990	1991
Total Equivalent Population Served with the Project	58,954	59,862	62,289	64,313	65,822	66,861	68,089	68,237	69,342
Estimated Sewage Discharge with the Project (MGD)[1]	8.58	8.7	9.05	9.34	9.56	9.71	9.88	9.91	10.08
BOD Loading (Tons/Year)	392	397	413	427	437	444	451	453	461
Induced Portion of Load (Tons/Year)	1	2	9	16	18	16	15	80	7
Suspended Solids Loading (Tons/Year)	392	397	413	427	437	444	451	453	461
Induced Portion of Load (Tons/Year)	1	1	9	16	18	16	15	80	7

Note: 1 Based on estimates in the Utilities EPTR.

The estimates in Table 3.5-28 are based on the assumption that secondary treatment levels will be maintained. If sewage facilities are overloaded, this may not be a valid assumption; however, quantifying the decrease in efficiency of a sewage treatment facility is not readily done unless extensive field data is available. Given the present poor quality of Crow Creek, this extensive program is not required.

3.5.3.2.3.2 Other Watersheds

Under the Proposed Action there are eight cities or towns that would receive induced population. This is the only factor that would cause altered wastewater discharges since the LF and LCFs will continue to discharge in the same manner as currently exists for the Minuteman III system.

The eight communities with induced population growth are: Pine Bluffs, Albin, and Kimball in the Lodgepole Creek Basin; Scottsbluff, Gering, and Torrington in the North Platte; and Wheatland and Chugwater in the Laramie Basin. The induced population growth occurs between 1985 and 1989. The peak years for the impact of this induced growth are given below:

Kimball	1989
Pine Bluffs	1988
Albin	1986
Scottsbluff	1988
Gering	1988
Torrington	1987
Wheatland	1986
Chugwater	1986

Table 3.5-29 shows the discharge flow and mass loading for BOD and TSS for each community during the peak year. This table is based on several assumptions. First, the zero discharge facilities will operate as designed. If this does not occur, the loads on Lodgepole Creek and the North Platte could be increased. Since Pine Bluffs discharges to a meadow, the surface water quality in Lodgepole Creek would show little or no change. Torrington would exert a load on the North Platte River that would amount to a 0.65 mgd (700 acre-ft/yr) flow at 30 T/yr BOD 5 and 30 T/yr TSS, compared to a baseline loading of 0.63 mgd (677 acre-ft/yr), 29 T/yr BOD_5, and 29 T/yr TSS. The second assumption used to develop these loads was that the per capita flow rates currently occurring will continue. Finally, the effluent quality achieved by all facilities is that required by the applicable National Pollutant Discharge Elimination System permit requirements.

The largest discharge impacts are in Kimball in 1989 and Wheatland in 1986 where 9.4 and 8.8 percent increases in sewage flow and pollutant loading, respectively, are expected. The other municipalities show less than a 2 percent increase over baseline conditions. The Kimball system has an

Table 3.5-29

PROJECT EFFECTS ON PEAK YEAR WASTEWATER LOADS IN OTHER WATERSHEDS

	Baseline Conditions					Induced Portion					Total				
	Population Equivalent	Flow mgd	Flow acre-ft/yr	BOD_5 T/yr	TSS T/yr	Population Equivalent	Flow mgd	Flow acre-ft/yr	BOD_5 T/yr	TSS T/yr	Population Equivalent	Flow mgd	Flow acre-ft/yr	BOD_5 T/yr	TSS T/yr
Kimball 100 gpcd 30 mg/l BOD 30 mg/l TSS	3,190	0.32	335	15	15	300	0.03	34	1.4	1.4	3,490	0.35	369	16.4	16.4
Pine Bluffs No Discharge	1,190	0	0	0	0	150	0	0	0	0	1,340	0	0	0	0
Scottsbluff 180 gpcd 30 mg/l BOD 30 mg/l TSS	14,860	2.57	2,871	116	116	210	0.04	45	1.8	1.8	15,070	2.61	2,916	117.8	117.8
Gering 160 gpcd 30 mg/l BOD 30 mg/l TSS	9,970	1.60	1,791	72	72	140	.02	22	0.9	0.9	10,110	1.62	1,813	72.9	72.9
Torrington[1] No Discharge	6,070	0	0	0	0	225	0	0	0	0	6,295	0	0	0	0
Wheatland 70 gpcd 30 mg/l BOD 30 mg/l TSS	4,820	0.34	380	16	16	450	0.03	34	1.4	1.4	5,270	0.37	414	17.4	17.4
Chugwater No Discharge	230	0	0	0	0	50	0	0	0	0	280	0	0	0	0
Albin No Discharge	130	0	0	0	0	10	0	0	0	0	140	0	0	0	0

Notes: 1 Torrington population includes South Torrington residents.
Values given below each city are parameters used for estimating sewage loads.

existing capacity of 0.58 mgd and Wheatland has a capacity of 0.5 mgd, therefore, those systems will not be overloaded during the peak year and no water quality problems should arise. If Torrington continues to discharge, the impact would be a 3.7 percent increase in loadings over baseline conditions. Since Torrington has initiated planning with regard to upgrading their sewage treatment facilities, it is expected that a zero discharge system could be constructed before the expected induced population growth occurs.

A 2 percent or lower increase over baseline loads is not expected to show any reduction in water quality for the watershed in question. Lodgepole Creek, with the greatest potential for change due to a 34 acre-ft increase over baseline, has a mean annual flow of 3,620 acre-ft/yr (in recent years). Therefore, the increase in sewage flow would cause less than an 1.0 percent change in flow in the Creek. The dilution would reduce the concentration of pollutants in the effluent by approximately a factor of five. Also, since Lodgepole Creek has exhibited increased flows further downstream, this dilution will continue to increase.

3.5.3.2.4 Surface Water Flow

The project is not expected to cause a major change in existing surface water flow patterns under average or low flow conditions.

3.5.3.2.4.1 Crow Creek Watershed

The proposed project is not expected to have significant effect on normal surface water flow patterns except as may be influenced by wells and increases in wastewater flow and resulting Crow Creek flow downstream of Cheyenne. Crow Creek may show some change in flow due to the possible increased pumping at the Cheyenne wellfield. At this time, it is not possible to quantify this change in flow and therefore not possible to assess changes in quality. However, pumping at the Cheyenne wellfield is not the only solution to the water supply question. If the water required for the induced demand is supplied through some other means, there will be no impacts on surface water flow in Crow Creek other than the discharge from the sewage treatment facilities.

The legal status of water rights associated with increases in municipal wastewater discharges return flows associated with imported (Cheyenne Stage II) water is not clearly resolved at this time. Increased municipal return flows may increase the amount of recharge to the groundwater aquifer; however, it is also possible that any increase in municipal return may be largely consumed by irrigated agriculture or partly used for urban irrigation. Wastewater reuse for urban irrigation would decrease surface water flow downstream of the treatment plants.

3.5.3.2.4.2 Other Watersheds

Removal of groundwater for use in the project will cause no measurable change in the surface water flow of any creek in other watersheds if well spacing is in accordance with Nebraska or Wyoming standards and at least 1,000 feet away from the streams.

Construction of cable paths, roads, and modifications of LFs in other watersheds will have no measurable change on surface water flow.

3.5.3.2.5 Modeling and Monitoring of Effects

No modeling or monitoring of surface water quality and/or hydrology should be necessary in watersheds outside Crow Creek watershed since impacts are anticipated to be small.

In order to determine the impact of well operations, monitoring may be necessary in Crow Creek upstream of Cheyenne. Such monitoring of surface water hydrology and water quality should be closely tied to any concurrent program for groundwater monitoring. Water level measurements, streamflow gaging, and water chemistry evaluations should be performed.

If any new wells were established in the DA, then site-specific monitoring of potential water level changes caused by the wells would be developed and implemented, as required.

3.5.4 Groundwater Hydrology and Quality

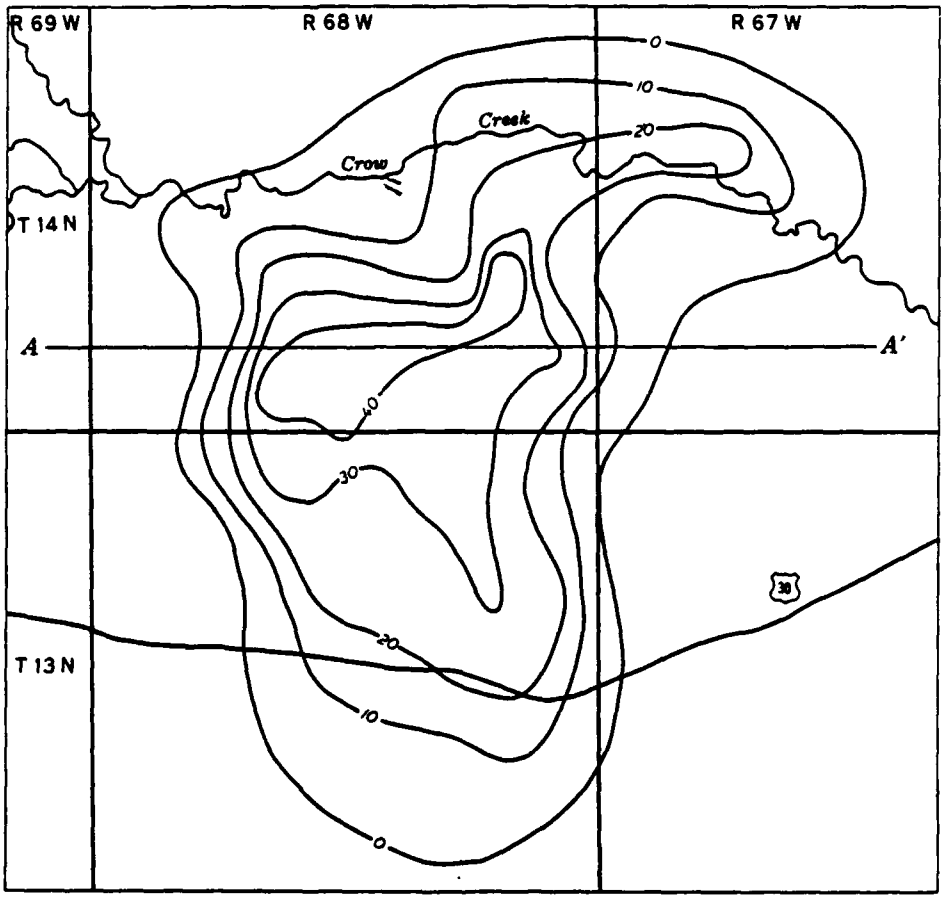
3.5.4.1 Baseline Future - No Action Alternative

3.5.4.1.1 Groundwater Hydrology

3.5.4.1.1.1 Crow Creek Watershed Area

A time comparison of limited potentiometric data for aquifers within the ROI from the early 1940s to 1977 suggests no significant (greater than 50 feet) long-term regional changes in water levels. Intensive development of groundwater in the Cheyenne wellfields has caused localized long-term overdrafts with a resulting decline in water levels. The pumpage from these wellfields has lowered the local potentiometric surface less than 50 feet (Figure 3.5-7). Equilibrium conditions at a new, but lower water level, were nearly established by 1964 (Figure 3.5-8). The volume of aquifer storage dewatered in the Cheyenne area was estimated to be 500,000 acre-ft (Lowry and Crist 1967). This lowering of the potentiometric surface has caused Crow Creek to become dry between Silver Crown (Section 17, T14N, R68W) and Section 18, T14N, R67W. Continued pumpage at the desired discharge rate of about 2,500 acre-ft/yr should not result in measurable additional lowering of the potentiometic surface nor additional effects on Crow Creek. Reduction of pumpage below this level would likely cause an increase in local water levels and a resumption of streamflow in the affected reach of Crow Creek.

Flow in Crow Creek downstream from Cheyenne has increased during the past decade. This increase in flow is the result of increased discharge from the Cheyenne area municipal treatment plants, and it is thought to be the cause of rising water levels in several wells located along the creek southeast of Cheyenne (Crist 1980).

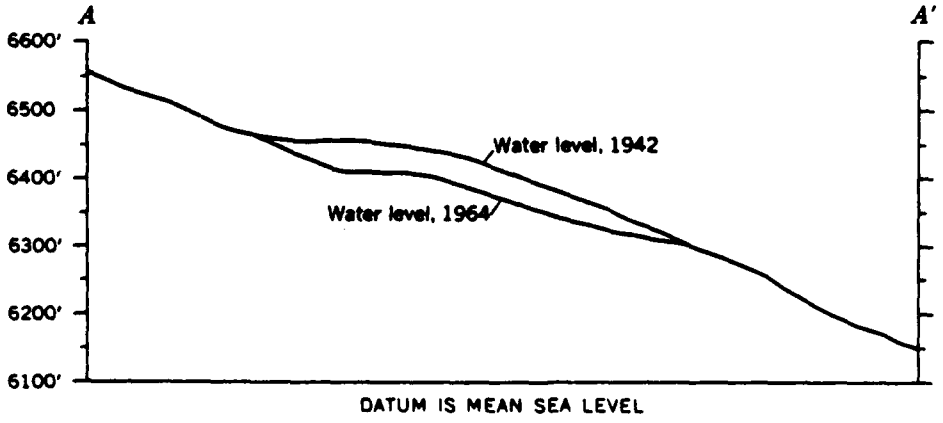


Base from Wyoming State Highway Department

Hydrology by Marvin A. Crist, 1965

SCALE IN MILES

0 1 2

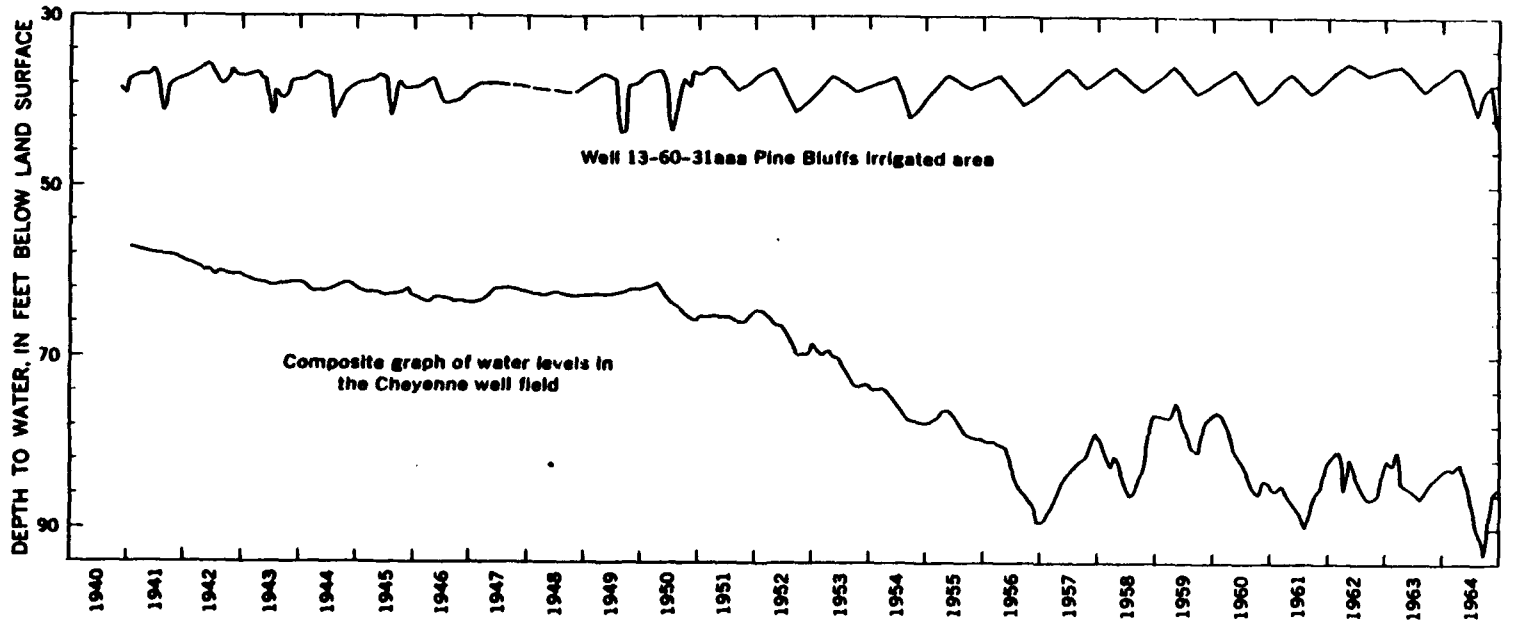


DATUM IS MEAN SEA LEVEL

(Adapted from LOWRY & CRIST, 1967)

AREA AFFECTED BY PUMPING & WATER LEVEL CHANGES IN CHEYENNE WELLFIELD

FIGURE NO. 3.5-7



(Adapted from LOWRY & CRIST, 1967)

LONG TERM TREND OF WATER LEVELS IN AREAS OF HEAVY PUMPING

FIGURE NO. 3.5-8

3.5.4.1.1.2 Other Areas

Water level declines of up to 40 feet have occurred in localized areas of Laramie County since 1920 as a result of groundwater pumping for irrigation (Crist 1980). Declines of 5 to 10 feet have occurred along Lodgepole Creek in Laramie and Kimball counties. Water levels have dropped about 5 feet since 1972 in Kimball, with large seasonal declines (Olssen Associates 1983).

Intensive development of groundwater in the Pine Bluffs area has caused interference between wells during the irrigation season and probably has decreased base flow in Lodgepole Creek (Crist and Borchert 1972). During the summer, water levels decline so much that some wells can be pumped only intermittently or at a reduced rate. Further development of groundwater from the post-Cretaceous aquifers in this area would aggravate the situation. Water levels in the Pine Bluffs area have exhibited seasonal and relatively long-term (1961 to 1971 period of record) declines (Crist and Borchert 1972). At the 1967 level of groundwater development, no permanent lowering of water levels is likely to occur (Lowry and Crist 1967). It is possible that continued pumpage at the 1971 rate will result in further reduction of base flow in Lodgepole Creek.

Continuous records are too short at most places to define more than a seasonal trend in water level fluctuations (Crist and Borchert 1972). Water levels in the Tertiary formations underlying sparsely populated areas are probably sensitive only to long-term trends in precipitation (Lowry and Crist 1967).

The general localized patterns of water level declines are projected to continue along with increased pumpage of groundwater. Additional supplies of water can be developed in most areas, but the potential for large yields is limited. Large yields with limited hydraulic effects on the post-Cretaceous aquifers may be available from the Casper Formation. The extent and nature of future hydrologic responses to groundwater pumping may be modified by groundwater management decisions at the state level.

Much of the area in Laramie and Kimball counties has experienced seasonal water level declines due to intensive development of groundwater. Water level declines of 10 to 20 feet occur in much of the area, with small areas greater than 30 feet. Further development of groundwater for irrigation purposes would likely aggravate this situation.

3.5.4.1.2 Groundwater Quality

3.5.4.1.2.1 Crow Creek Watershed Area

Increased discharges from the Cheyenne Urban Area wastewater treatment plants could adversely affect the quality of the groundwater in the area downstream of Cheyenne. Elevated nitrate and TDS levels have been found in that area, and this front may move downstream as a result of increased discharges.

Using the simple mass balance described in Section 2.6.4.4, nitrogen and TDS levels would be forecast to be 4.7 mg/l and 308 mg/l downstream of Cheyenne in 1987. This is based on an assumed total recharge of 22,150 acre-ft/yr. This assumes that one-half of the 10,300 acre-ft/yr of wastewater effluent recharges the groundwater basin and has the same concentrations as assumed for

1980. The results indicate water quality standards will still be met, but the accuracy of the results in representing the physical system is unknown.

No other changes in groundwater quality are foreseen based on past trends. Limited data are available to define temporal or spatial trends in water quality in the post-Cretaceous aquifer. Analysis of water from the Cheyenne wellfield in 1947 and 1964 show very little change in chemical composition.

3.5.4.1.2.2 Other Areas

Higher than expected nitrates in some wells in the remaining portion of the ROI are due to chemical and organic fertilizer, substandard well-septic tank spacing, and livestock population. Elevated nitrate levels are quite localized. These levels can be expected to remain elevated.

Lowry and Crist (1967) report increasing TDS levels in one well completed in Quaternary materials in the Pine Bluffs area. They consider this increase to be due to irrigation. Few wells have multiple analyses reported and therefore this trend of increasing TDS cannot be confirmed.

3.5.4.2 Proposed Action

3.5.4.2.1 Interference with Springs, Wells, and Surface Flow

Water supply requirements for induced water demand and construction purposes can be met by several water management plans or combinations thereof. In order to evaluate the effects of each option, certain assumptions have been made. These assumptions and effects will be discussed for both the Crow Creek watershed and the remaining areas within the ROI.

3.5.4.2.1.1 Crow Creek Watershed Area

Water to meet induced demands could be supplied by existing wells in the Cheyenne wellfields. This may require the development of specific pumping patterns or schedules for existing wells or relocation of some wells to minimize the effects of increased pumpage. Rehabilitation or relocation of some wells may also be necessary to obtain the required quantities of water.

Historically the CBPU has attempted to operate its wellfields to meet peak demands. Consequently, the rate of withdrawal from the groundwater sources has varied widely from year to year, with a long-term average discharge rate of 3,400 acre-ft (1941 through 1978). Lowering of water levels with the consequent reduction of spring and streamflow are documented in the Cheyenne vicinity as a result of water withdrawal from the Cheyenne wellfields (Lowry and Crist 1967, USFS 1981).

As an example, during the drought years of 1977 and 1978, the wellfields were pumped almost continuously at full production rates. A total of about 13,100 acre-ft was removed during that period. As a result, water levels locally decreased approximately 20 feet and maximum pumping capacity of the system decreased from 800 to 500 acre-ft per month (Ertec Western 1983b).

If the wellfields were to be pumped at the long-term average rate of 3,400 acre-ft/yr, the water table could be expected to decline at a rate of 3

to 5 feet per year. This is based on historic changes in water level as a response to pumping.

Past performance of the wellfields indicates that a withdrawal rate of about 2,500 acre-ft/yr can be sustained without causing a continuing decline of the phreatic surface (USFS 1981). During the construction period of 1984 through 1990 there is only 1 year (1984) in which demand for the Cheyenne Urban Area could not be met without pumping at a rate greater than this. Cheyenne will have a shortfall of approximately 3,970 acre-ft in 1984 due to construction of the Stage II Diversion Project. With the Proposed Action this would increase to 4,040 acre-ft. In order to meet this shortage it is likely that the CBPU will pump between 4,000 and 7,000 acre-ft from its wellfields. This could result in water level declines in excess of 5 feet; however, the incremental amount attributable to the project-induced demand of 70 acre-ft is minor.

A response to the historical lowering of water levels has been the decrease in surface flow in Crow Creek in the vicinity of the Cheyenne wellfields. The magnitude of this effect is such that the stream reach between Silver Crown and the wellfield no longer has perennial flow (Lowry and Crist 1967). It is likely that the length of the ephemeral reach would be increased if induced water demands are supplied from the Cheyenne wellfield.

A numerical groundwater model of the Middle Crow Creek watershed has been developed to assess the potential for increased or modified groundwater development in the area of the municipal wellfields. This localized model, developed by the Earth Technology Corporation (1983b), is based on the regional groundwater model of Laramie County developed by the USGS in cooperation with the Wyoming State Engineer's Office (Crist 1980).

Using the numerical model, an 11-year period (1983-1993) with annual base pumpage of 2,425 acre-ft was simulated in monthly time steps. The pattern of pumpage simulated reflected recent use and distribution of pumping between existing wells. The resulting drawdowns were compared with those simulated after imposing the induced demands due to the Proposed Action on this baseline pumpage for the same period. This assumes that the entire Cheyenne induced demand would be met with the municipal wells. Maximum drawdown was found to occur at the end of October for each year of simulated pumpage. At the end of October 1993 the maximum difference in simulated water levels (model node with maximum drawdown increase) between baseline pumpage and baseline-plus-potential project pumpage was approximately 3.5 feet of additional drawdown at the point of maximum drawdown.

Subsequently, pumpage amounts were extended for the estimated life of the project (30 years total) and resulting drawdowns were compared. At the end of October 2013, the maximum simulated difference in water levels between the baseline pumpage and baseline-plus-project pumpage was only slightly more than the previously simulated difference of 3.5 feet of additional drawdown.

As another option, project-induced demands could possibly be satisfied by water obtained from that portion of the Casper Formation underlying the eastern flank of the Laramie Range approximately 20 miles north of Cheyenne. This extends into the Lodgepole and Horse Creek watersheds.

The water yield potential and the quality of the Casper Formation on the east side of the Laramie Range is essentially undefined. Based on general hydrogeologic considerations it is generally assumed to be a viable although limited water supply source. A reconnaissance level evaluation of the water supply development potential of the Paleozoic aquifer system has been conducted by the Earth Technology Corporation (1983a). Results indicate that yields in excess of several hundred acre-ft/yr are unlikely.

To avoid effects on the surface water system the aquifer would be developed at intermediate depths (1,000 - 3,000 feet) and at a sufficient distance from the areas where the formation outcrops and is interconnected with the surface hydrologic system. Because this aquifer is not currently utilized within Laramie County, development of water supplies would not affect existing water rights.

A third option for meeting project-induced demands in the Cheyenne Urban Area and Crow Creek watershed is to purchase or lease existing groundwater rights. If project-induced demands are met by water supplied by this source, it will have no effect on the hydrologic system in the area due to the State of Wyoming's regulations governing sale or lease of groundwater (Section 2.6.2).

3.5.4.2.1.2 Other Areas

For areas outside of the Crow Creek watershed within the ROI, water will be needed for construction in the DA and will also be required for domestic use for immigrants living in Pine Bluffs, Albin, Torrington, Cheyenne, and Wheatland, Wyoming, and Scottsbluff, Kimball, and Gering, Nebraska.

It is possible that construction water could be obtained from the existing wells at each of the Minuteman LCFs. These wells were constructed in 1962 and little is known about their current condition. Since they are unable to meet the anticipated peak construction demands of about 100 gpm, the wells would have to be enlarged, new wells developed, or existing rights purchased or leased. Any of these three alternatives would have to be done in accordance with the appropriate state water laws.

The effect of increased pumpage on water table levels can be estimated by using the Hantush-Jacob equation (Hantush-Jacob 1955) for unsteady flow to a well in a leaky confined aquifer. The equation is presented below:

$$S = \frac{Q}{4\pi T} \quad W(u, r/B)$$

where:

$$u = \frac{r^2 S}{4Tt}$$

$r/B = r\sqrt{K^1/Tb^1}$
s = drawdown of an observation well at a distance r from the pumped well
Q = constant well discharge
T = transmissivity of the aquifer
S = storativity of the aquifer
t = time since pumping started
K^1 = vertical hydraulic conductivity of the aquitard
b^1 = thickness of the aquitard

W(u,r/B) is know as the leaky well function.

The assumptions involved with using this equation are:

- The aquifer is homogeneous, isotropic, infinite in areal extent and is of uniform thickness throughout.
- The well completely penetrates the aquifer, and flow in the aquifer is radial throughout.
- The aquifer is overlain by an aquitard of low but finite permeability which is in turn overlain by an unconfined aquifer. The aquifer is underlain by an aquiclude. Flow in the aquifer is augmented by vertical leakage through the aquitard.
- No water is released from storage in the aquitard.
- The overlying water table is not influenced appreciably by pumping.

Not all of these assumptions are met by the post-Cretaceous aquifers in the ROI. However, when drawdown is not great and recharge occurs, this equation can be calibrated to a vertical hydraulic conductivity and used to approximate drawdowns with time as a result of pumping. The vertical hydraulic conductivities were calibrated to the steady state drawdown derived from the Weber technique (Keck 1976).

The equation used in the Weber technique is defined as follows:

$$s^1 - \frac{(s^1)^2}{2m} = \frac{Q}{2\pi T} \left[\ln \left(Q/\pi r^2 w\right) + \pi r^2 w/Q - 1\right]$$

Where: s^1 = drawdown observed in unconfined aquifer
m = initial saturated aquifer thickness
Q = well discharge
T = aquifer transmissivity
w = rainfall accretion

The results from applying these equations to the Ogallala and Arikaree aquifers using values typical of those aquifers in the area are presented in Table 3.5-30.

The results shown are for constant pumping at the given rates. Because constant pumping will not occur during the actual construction, these estimates of drawdown are greater than what would actually occur. The magnitude of the drawdown would depend on site-specific hydrologic characteristics, well construction/efficiency, and construction water demand schedule.

Because existing wells are not densely concentrated throughout the ROI, water level declines in existing wells are expected to be small. The effects on the groundwater system in the ROI due to construction water requirements will decrease after the period of construction when water requirements decline.

Major streams in the ROI are both gaining and losing. Groundwater extraction from post-Cretaceous aquifers in the vicinity of these streams may locally lower water levels to the extent that groundwater discharge to streams is reduced. The potential for this is a function of local hydrologic parameters and the distance from the well to the stream.

Domestic water must be provided for inmigrants moving into the towns of Pine Bluffs, Kimball, Gering, Scottsbluff, Torrington, Albin, Chugwater, and Wheatland. Because all these municipalities obtain their water from wells, an increase in water demand could affect groundwater levels which in turn could affect surrounding wells or flow in streams in contact with the aquifers involved. To estimate the effects of this induced water demand, the increase in water demand was compared with baseline water demand to calculate the percentage increase involved. The induced demand as a percent of baseline demand would be less than 2 percent in the peak year for Albin, Torrington, Gering, and Scottsbluff. The largest percentage induced demand will occur in 1985 in Chugwater when the induced demand will be about 35 percent of the baseline demand. In Wheatland, Pine Bluffs, and Kimball the increase will range from 6 to 8 percent in the year of peak-induced impact. All communities currently have sufficient water supplies to meet these demands and increased withdrawal rates should not appreciably increase drawdowns due to the relatively small quantities of water involved. The City of Kimball is currently planning to add another well to its system to reduce daily water level fluctuations and impacts on neighboring irrigation wells. Even if the system is not expanded, the induced demand should not cause a noticeable increase in short-term water table variations and will not cause an increase in any long-term water table declines. The induced demands will also not affect the availability of water to neighboring wells.

Table 3.5-30

EFFECT OF NEW DEPLOYMENT AREA WELLS ON WATER LEVELS 1,000 AND 5,000 FEET FROM WELL PUMPING 100 GPM

	1,000 Feet	5,000 Feet
	(Feet of Lowering)	
Ogallala Aquifer		
After 1 Year	1.5	0.02
After 5 Years	1.9	0.03
Steady State	1.9	0.03
Arikaree Aquifer		
After 1 Year	3.0	0.01
After 5 Years	5.2	0.05
Steady State	5.8	0.08

Note: Based on a specific yield value of 0.15 and transmissivities of 2,000 sq ft/day for Ogallala aquifer and 700 sq ft/day for Arikaree aquifer.

3.5.4.2.2 Wastewater Discharges

3.5.4.2.2.1 Crow Creek Watershed Area

In the Crow Creek watershed, three wastewater treatment plants in the Cheyenne Urban Area discharge to Crow Creek. The effects on surface water quality are discussed in Section 3.5.3.2.3.1. Because Crow Creek is a losing stream downstream of the Cheyenne Urban Area, the quality of the groundwater is affected by these wastewater discharges.

It is likely that any increased effluent discharge into Crow Creek would cause the localized elevated level of nitrates and TDS to move further down gradient.

The simple mass balance evaluation applied to 1987 with project conditions would indicate a nitrogen level of 4.9 mg/l and a TDS level of 312 mg/l downstream of Cheyenne. These are increases of 0.2 mg/l and 4 mg/l, respectively, over baseline conditions. The results indicate water quality standards will continue to be met.

3.5.4.2.2.2 Other Areas

There are several wastewater disposal facilities in the remaining ROI which could adversely affect the quality of the groundwater. They may be grouped as follows:

- Municipal facilities discharging to surface water;
- No discharge lagoons owned by municipalities or by the Air Force;
- Septic tanks followed by lagoons, owned by the Air Force; and
- Sump discharges at the LFs.

The effects of each of these types of facilities will be discussed separately.

The cities of Wheatland, Gering, Kimball, and Scottsbluff all own treatment facilities which discharge to surface waters. In the areas in which the facilities discharge, the streams are all gaining streams, i.e., they gain water from the aquifers. Because the streams do not discharge to the groundwater, an increase in discharge from the wastewater treatment plant will not affect the quality of the groundwater.

No discharge lagoons treat wastewater generated in Pine Bluffs and Torrington and at LCFs in Flights R, S, A, B, and C. These facilities do not discharge to surface waters; however, any water that is not evaporated from these ponds seeps into the ground, where degradation of the groundwater may occur. Elevated nitrate levels are one possible result of wastewater seeping from lagoons into the groundwater. No water quality problems have been reported in the area of the lagoons. Increased flow to the lagoons due to inmigration or construction activities at the LCFs could cause an increase in seepage to the groundwater. However, the effects of increased seepage on groundwater quality are expected to be negligible.

Sewage treatment is accomplished by a septic tank/lagoon combination at LCFs P, Q, T, D, and E. The same possibility of groundwater degradation due to increased flow exists at these Flights as at those listed above. As with those lagoons, no problems have been reported and the effects of the increased flow are expected to be minimal.

At the present time, there are no toilet facilities at any of the LFs. The maintenance crews are supposed to have small portable units with them if they will be in the field for a long period, otherwise, they use the LCF facilities. Portable sanitary facilities will be provided when construction activities commence at the LFs. These will not adversely affect the quality of the groundwater.

As discussed in Section 2.0, water pumped from the sumps of the missile silos may affect groundwater quality at the LFs. There is not expected to be an increase in the amount of water pumped over present conditions. Therefore, there will be no increased degradation of the groundwater from sump pumping as a result of this project.

3.5.4.2.3 Subsidence and Seismicity

3.5.4.2.3.1 Crow Creek Watershed

It is not anticipated that subsidence would occur in the Crow Creek watershed as a result of groundwater withdrawal for the project. The deposition and subsequent erosion of the post-Cretaceous formations has left these formations in a compacted state, such that the additional water removed from these formations should not lead to additional compaction and subsidence. The potential for induced seismicity in the Crow Creek watershed is also low.

3.5.4.2.3.2 Other Areas

The potential for subsidence or induced seismicity in the remainder of the ROI is, as is the case in the Crow Creek watershed, very low.

3.5.4.2.4 Modeling and Monitoring of Effects

Analytic methods should be used to evaluate hydrologic effects based on site-specific characteristics at each potential producing well. The hydrologic characteristics should be evaluated by test pumping. Data from fully penetrating monitor wells and stream gages would be required to evaluate the aquifer and stream effects.

Computer modeling of the Cheyenne wellfields is being done to evaluate hydrogeologic parameter spatial sensitivity and simulate possible changes in water level and surface water depletion. Alternative operational plans and potential well relocations and rehabilitations are also being evaluated as part of water supply development planning for the project (Earth Technology Corp. 1983b).

3.6 Summary of Impacts

This section presents a summary of impacts expected for the Proposed Action and project alternatives based on the analysis of Section 3.5 and the level of impact definitions and significance determination criteria presented in Sections 3.3 and 3.4.

3.6.1 Impact Matrix

Figure 3.6-1 provides a summary assessment of the level and significance of impacts to the water resource system of the Proposed Action for the various water resources elements and subelements. Significant, short-term, moderate impacts are expected in the Cheyenne Urban Area due to induced water demands, storm runoff, and erosion and siltation. These project changes are significant since they could interfere with existing users or endanger public health or safety and are rated moderate since it may be necessary to construct minor new facilities to mitigate impacts. All other project impacts are rated not significant because with assumed mitigations they would not violate laws or regulations adopted to protect or manage the water resource system, endanger public health or safety, reduce water available to or interfere with existing users, or change preproject hydrologic baseline conditions over an extensive area or period of time. There are several components of the Proposed Action for which alternative actions have been identified. These include the following:

- The cable routes - Five routes of 11 possible routes constitute the Proposed Action.
- Three alternative access roads at F.E Warren AFB; and
- Alternative construction dispatch areas - The Proposed Action includes dispatch stations at Chugwater and Kimball. One alternative has a station at Kimball only. The third alternative has no dispatch stations.

Figure 3.6-2 summarizes the level and significance of impacts on water resources for each element of these components, including those contained in the Proposed Action. None of the project alternatives would alter impact assessments made for the Proposed Action. The following sections discuss the impacts of the Proposed Action and of these alternatives by water resource element.

For purposes of the the discussion the following definitions apply:

Site	Local	Regional	
Crow Creek Watershed	F.E. Warren AFB Urban Area	Cheyenne	Crow Creek watershed

LEGEND		ADVERSE IMPACTS	SIGNIFICANT ADVERSE IMPACTS
LEVEL OF IMPACT*	LOW	o	●
	MODERATE	○	●
	HIGH	○	●
POTENTIAL BENEFICIAL EFFECTS			

* MEASURE OF THE AMOUNT OF ENVIRONMENTAL CHANGE

	CROW CREEK WATERSHED						OTHER AREAS					
	SHORT TERM			LONG TERM			SHORT TERM			LONG TERM		
	SITE	LOCAL	REGIONAL	SITE	LOCAL	REGIONAL	SITE	LOCAL	REGIONAL	SITE	LOCAL	REGIONAL
Water Use and Demand												
Construction Water Demand	o						○	○				
Induced Water Demand	o	●		o	o			o				
Constraints on Water Use												
Changes in Constraints												
Changes to Individual Water Rights		o					o	o				
Surface Water Hydrology & Quality												
Storm Runoff and Drainage	o	●	o	o[1]	●[1]	o[1]	o			o[1]		
Erosion and Siltation	o	●	•				o					
Wastewater Discharges		○	o		o	o	o	o				
Surface Water Flow												
Ground Water Hydrology & Quality												
Interference with Springs, Wells, and Surface Flow		o						o				
Regional Groundwater Quality												
Wastewater Discharge Effects on Water Quality		o	o		o	o	o	o				

[1] Impacts are those generated by construction activities and having a long duration.

DETAILED IMPACT MATRIX

FIGURE NO. 3.6-1

LEGEND

LEVEL OF IMPACT Measure of the Amount of Environmental Change		
	Adverse Impacts	Significant Adverse Impacts
Low	o	●
Moderate	○	●
High	◯	●
Potential Beneficial Effects	▒	

	CABLE PATHS											ROADS			DISPATCH STATIONS		
	PA1	SB1	RB1	PA4	PD1	SB2	PB1	PA5	PA2	PA3	RB2	R1	R2	R3	PROPOSED	ALT.	NONE
Water Use and Demand																	
Construction Water Demand	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	
Induced Water Demand																	
Constraints on Water Use																	
Changes in Constraints																	
Changes to Individual Water Rights																	
Surface Water Hydrology & Quality																	
Storm Runoff & Drainage																	
Erosion and Siltation												o	o	o			
Wastewater Discharges																	
Surface Water Flow																	
Ground Water Hydrology & Quality																	
Interference with Springs, Wells and Surface Flow																	
Regional Groundwater Quality																	
Wastewater Discharge Effects on Water Quality																	

WATER RESOURCES ALTERNATIVES COMPARISON MATRIX

FIGURE NO.3.6-2

Site	Local	Regional	
Other Watersheds	Within 1,000 feet of silos, roads, or cables	Within 2 miles of silos, roads, cable routes, or municipalities	Watershed in which impacts occur

3.6.1.1 Water Use and Demand

3.6.1.1.1 Construction Water Requirements

3.6.1.1.1.1 Crow Creek Watershed

Site. The only construction water requirement within the Crow Creek watershed is for use at F.E. Warren AFB. Baseline water demands at the base are projected to remain constant at 1,070 acre-ft/yr. Construction water use at the base with the Proposed Action will increase demands by 13, 49, and 5 acre-ft/yr, respectively, in 1984 through 1986. Although this is a relatively minor increase, this water will be purchased from Cheyenne, which is projected to have a water deficiency in 1984 under the Proposed Action and baseline conditions. Because the base system capacity will not be exceeded and would only require some operational increases, this use constitutes a low impact in the short term. There are no long-term construction water demands at F.E. Warren AFB, therefore, this impact is rated negligible in the long term. This amount of water use would not interfere with existing users so the impact is rated not significant.

The total amount of water required for construction of roads at F.E. Warren AFB would be approximately 8 acre-ft including contingencies. This would vary slightly for the three alternative routes. Alternative R1 would require slightly less water based upon the miles of road to be constructed; however, this difference would have minimal effects upon the amount of construction water required. Based upon this all, three alternatives are rated as having a low, not significant impact.

Local. There are no short or long-term construction water requirements for the Cheyenne Urban Area other than the construction of offbase roads which would use a small amount of water. The impact of construction water demand on the Cheyenne Urban Area is rated negligible for both the short and long term and since it would not interfere with existing users, is rated not significant.

Regional. There are no short or long-term construction water requirements in the Crow Creek watershed in the area surrounding the Cheyenne Urban Area. Therefore, these impacts are rated negligible.

3.6.1.1.1.2 Other Areas

Site. For this analysis of impacts it has been assumed that construction water for the DA, i.e., LFs, LCFs, and DARs would be supplied by new groundwater appropriations. This would entail the construction of new wells and would therefore constitute a moderate impact. Because water acquisition will comply with existing regulations and use would be relatively minor

compared to existing use (160 acre-ft and 88 acre-ft in the peak years throughout the Wyoming and Nebraska portions of the DA), the impacts would not be significant. There are no long-term construction or operational water requirements in excess of existing conditions and long-term impacts are negligible. If water requirements for construction were obtained from existing appropriations, impacts would be less.

Revegetation of the cable rights-of-way after construction could require the application of approximately 6 inches of water. Assuming that the equivalent of a 10-foot wide area along the entire route requires revegetation, water requirements would be 0.6 acre-ft per mile. Based upon these assumptions there are varying construction water demands for the alternate cable routes. These are shown below and are included in the contingencies shown in earlier demand projections.

Cable Route	PA1	SB1	RB1	PA4	PD1	SB2	PB1	PA5	PA2	PA3	RB2
Water Required (acre-ft)	0.6	7.2	15.0	13.2	16.8	15.6	17.0	14.4	7.8	8.4	8.4

For the Proposed Action the vegetation requirements are about 50 acre-ft. The minimum water requirement using alternative routes would be about 32 acre-ft. The alternative routes with the maximum requirement would use about 68 acre-ft. All of these constitute low, not significant construction water impacts since the amount used in any given area is very small.

Local. It has been assumed that some new construction wells may be located away from actual construction areas due to constraints on water availability. Construction wells would constitute a minor new facility and impacts would be moderate, though not significant, using the same reasoning as for wells at the actual construction sites. There are no long-term demands; impacts are therefore negligible in the long term.

For both of the above areas, if construction water were to be supplied from existing wells, i.e., irrigation or municipal wells, then the impact would be rated as low and not significant in the short term because existing facilities would be capable of supplying the construction water.

The project alternatives regarding DAR roads and communication cables may change the location of impacts, but the impacts would still be low and not significant. Having no dispatch station in Kimball or Chugwater would slightly reduce water demands there.

Regional. There would be no short or long-term construction water use in areas within the ROI removed from the DA, e.g., the Scottsbluff area. The impacts are therefore negligible for these areas.

3.6.1.1.2 Induced Water Demands

3.6.1.1.2.1 Crow Creek Watershed

Site. There will be an increase in water demand at F.E. Warren AFB beginning in 1985 due to the operations of new facilities and the additional operations personnel required. This will peak at 92 acre-ft/yr in 1987 and decline to a long-term demand of 76 acre-ft. This demand can be supplied by the CBPU system in all project years, although a slight shortage in 1991 is projected with the Proposed Action. The impact is rated as low in both the short and long term. Because Cheyenne will require additional water supplies in 1992 without the project, this demand at the base is rated not significant, since its quantity and timing will not cause interference with existing users.

Access road construction is a small portion of the total construction required at F.E. Warren AFB for the Proposed Action. The various alternative routes would have no impact upon the number of inmigrants to the area for project construction. Therefore, the impact of each alternative is rated as negligible.

Local. Under the Proposed Action the short-term (1984-1990), project-induced increases in demand can be met by the CBPU system in all years but 1984. The projected baseline shortage of 3,970 acre-ft in 1984 will be increased by 70 acre-ft with the project. In other years demands can be met by the existing system; however, in order to assure that the storage reservoirs are adequately refilled following the 1983 to 1984 shortage period it will be necessary to operate the municipal wells to provide 2,000 acre-ft/yr and to meet peak summer demands. Groundwater modeling by Earth Technology Corporation (1983b) and discussions with CBPU personnel indicate that in order for the wells to perform at this level during the construction period, some rehabilitation and perhaps relocation of some of the wells will be required. Because of this potential for construction of minor new facilities and operational modifications this impact is rated as moderate. Although no substantial reduction in availability of water to existing users is foreseen, some interference could occur; therefore, the impact is rated significant in the short term.

The long-term induced demand on the Cheyenne water supply is about 250 acre-ft from 1991 onward. Water would not be available to meet this demand in 1991 and operational modifications or additional raw water supplies would be required. Under the No Action Alternative, additional raw water supplies would be required in 1992, rather than in 1991 as in the Proposed Action. This is rated as a low, not significant impact.

Regional. There would be no project-induced increase in water use in the Crow Creek watershed outside of F.E. Warren AFB and the Cheyenne Urban Area. Therefore, these impacts are negligible and not significant, both in long and short term.

3.6.1.1.2.2 Other Areas

Site. There would be no induced water demands to support construction personnel at the construction sites in the DA. Both long and short-term impacts would be negligible.

Local. For purposes of this analysis, the local area is assumed to include the communities in other ACSs. Several communities would experience some project-induced population growth and resulting increases in water use. These increases range from less than 1 percent to 35 percent. All of these communities have sufficient capacity to supply these increased demands with slight increases in groundwater pumping rates. The short-term impact is therefore ranked as low due to this change in operations, and not significant since it would not interfere with existing users.

There are no long-term population increases projected for any of these communities due to the Proposed Action. Since there will be no permanent increase in water demand due to the project, this impact is negligible in the long term. The selection of cable routes would have no effect upon the distribution of population within the ROI. Therefore, there is no relationship between the cable route alternatives and induced water demand.

Regional. There would be no project-induced demand in other ACSs outside the above areas. Therefore, the impacts are negligible and not significant, both long and short term.

3.6.1.2 Constraints on Water Use

3.6.1.2.1 Changes in Constraints

Throughout the ROI, controls on water use and water quality controls on discharges to the water resource system are not expected to change from present conditions, so negligible and not significant impacts are forecast. This applies to both the short and long term. No changes in constraints are expected to occur because of the installation of communication cables, road construction, or dispatch center establishment.

3.6.1.2.2 Changes to Individual Water Rights

3.6.1.2.2.1 Crow Creek Watershed

Site. It is not expected that any water rights would need to be acquired at F.E. Warren AFB, either short or long term. Therefore, the impacts are negligible and not significant. Water for road construction would most likely come from the CBPU water system. Therefore, no water rights would be changed and there will be no impacts.

Local. One alternative to meet the induced demands on the CBPU water supply is to purchase or lease existing water rights. Such a lease or purchase would have low, not significant, short-term impacts and negligible, not significant impacts in the long term.

Regional. Throughout the remainder of the watershed, it is not expected that any water rights will change hands as a result of the project. Impacts would be negligible and not significant, both long and short term.

3.6.1.2.2.2 Other Areas

Site. Water will be required in the short term at the LCFs for construction purposes. Because it is not known whether the existing wells at the LCFs can provide sufficient water, it may be necessary to lease or purchase groundwater rights in Wyoming or to develop new wells at those LCFs located in Nebraska. Any of these alternatives would have low, not significant impacts in the short term and negligible, not significant impacts in the long term.

Water would be required for revegetation after cables are installed. Water would be acquired from existing wells or from wells already appropriated for construction uses. The incremental amount needed for cable route revegetation would be minimal. Therefore, the impacts are negligible and not significant.

Local. As discussed in the previous section, construction water will be required and it is not known whether the existing wells at the LCFs can provide adequate amounts of water. If the wells which provide the construction water, whether they are leased, purchased, or newly developed, are located off the LCF site, the impacts could be local in scope. The impacts would be, in the short-term, low and not significant and negligible and not significant in the long term.

Regional. In the remainder of the ROI, no acquisition of water rights is expected to take place, either short or long term, as a result of this project. Negligible, not significant impacts would occur.

3.6.1.3 Surface Water Hydrology and Quality

3.6.1.3.1 Stormwater Runoff and Drainage

3.6.1.3.1.1 Crow Creek Watershed

Site. The site impacts due to stormwater runoff and drainage are rated as low and not significant on F.E. Warren AFB. The rating of low is based on development of new land and 50 additional acres of impervious area that would increase site runoff from 22 to 58 cfs over a large area for a 50 year, 6 hour storm and would not likely require drainage improvements for any major drainageway. Assumed mitigations should control site runoff on the base to create the low and not significant impact rating. Small but measurable changes in flow will occur for both the short and the long-term cases; slight modification to existing drainage systems will probably be necessary near this area of development change.

The three road system alternatives have all been determined to have a negligible impact. The three systems are new roads or upgraded versions of existing roads. Stormwater runoff and drainage facilities for the existing roads will be able to accommodate any increased flow. The impact is also determined to be not significant because there is no danger to public health or safety and no laws will be violated.

Local. Analysis of runoff events in the Cheyenne Urban Area indicates no major change in flows in any drainage channel due to baseline or impact growth. The greatest change would be in Dry Creek where a 1.0 percent increase in drainage channel flow could occur by 1987 without the project and

a 1.8-percent increase in drainage channel flow could occur by 1987 under project conditions for a 50 year, 6 hour storm. These increases create a low and not significant impact since modifications to accommodate increases would not require new facilities and no danger to public safety would occur. However, overall storm runoff and drainage impacts felt in the Cheyenne Urban Area (local) are rated as moderate and significant for both the short and long term. Localized increases in peak storm flows due to development will produce measurable changes in flood flows which will require modifications in operations of existing facilities or construction of minor new facilities. These facilities would be needed to safely transfer increased flow from the area of development to the major drainage channels. Additional storm sewer capacity or detention storage will need to be developed. This impact is considered significant due to the density of the population in the area. The level of impact could be reduced to low by requiring lower density housing and planned development. The significance of the impact could be reduced by not allowing development in flood-prone areas. The impact would be low and not significant if City of Cheyenne requirements relative to storm drainage were met.

Regional. There will be no increased impervious area in the Crow Creek watershed, excluding site and local areas, but measurable changes in runoff could occur. The project could cause a 2-cfs (0.02 percent) increase in Crow Creek flow for a 50-year, 6-hour storm event. This level of increase should not require drainage improvements. Therefore, the project impacts on stormwater runoff are rated as both low and not significant.

3.6.1.3.1.2 Other Areas

Site. The site impact level for LFs, LCFs, and upgraded roads are determined to be low for the short and long term. Short-term impact is due to disturbance in the LF or road areas during construction. Temporary site drainage facilities will have to be modified for increased short-term runoff. Long-term impacts are negligible because of only very minor permanent increases in impervious area and revegetation of disturbed areas. In neither case is the impact significant, no laws will be violated, and at no time is public health or safety in jeopardy.

The cable routes will not have any impact on surface water runoff and drainage; consequently all cable alternatives are given the designation N/A for not applicable.

Local. Local stormwater runoff and drainage impacts in the other watersheds are determined to be negligible for both the short and long-term cases. No easily measurable changes in flood flows on a local level are expected as a result of the project. This impact is also determined to be not significant for both cases.

Regional. No impact, on a regional basis, is expected to occur as a result of the project. On a watershed level, no measurable increases in stormwater flow are expected. The impact is not significant for both long and short term because no laws would be violated.

3.6.1.3.2 Erosion and Siltation

3.6.1.3.2.1 Crow Creek Watershed

Site. It is estimated that there could be an increase in erosion of 9,900 tons on F.E. Warren AFB during construction without mitigation. This constitutes an impact of moderate proportion requiring changes in operations or minor new facilities. However, assumed mitigation measures to be implemented during construction would reduce both the extent and effect of erosion. The site-level impact is therefore rated as low. Long-term impacts are expected to be negligible once revegetation of disturbed soils has occurred.

Alternative access roads to F.E. Warren AFB have varying erosion potential. The variation would be slight based upon the miles of road to be constructed, with Alternative R1 causing the least erosion. Based upon this, all three alternatives are rated as having low impacts in the short term. Long-term impacts are expected to be negligible.

All of these alternatives would have a low impact because they slightly increase erosion. However, by using good sediment control, accepted Air Force construction procedures, and revegetation, no health or safety risks, or changes from preproject basin characteristics would occur nor would laws be violated. The impacts are therefore not significant.

Local. Construction due to road work and induced by population growth due to the project is expected to increase erosion by 7,000 tons in the peak year of 1985. Because erosion and siltation can be controlled by operational controls or construction of minor facilities, this constitutes an impact of moderate proportion. It is expected that this could be reduced to a low level of impact if procedures comparable to the assumed mitigation on F.E. Warren AFB were implemented. This impact is estimated to be significant in that it will degrade water quality in Crow Creek and its tributaries. Sedimentation would also increase, which would reduce channel carrying capacities and increase the likelihood of flooding.

Long-term impact at the local level is expected to be negligible. Erosion at construction sites would not be expected to remain high once revegetation or coverage with impervious material has occurred. Any long-term change in erosion potential would be slight and would not be significant.

Regional. Erosion and sediment delivery to the Crow Creek watershed is estimated to increase 0.7 percent in the peak year due to project development. Since mitigation would require operational changes, i.e., dredging, this is a low impact. The regional impact is caused by changes in the local area and construction of minor facilities may be required there to mitigate regional impacts. The impact is significant in that water quality is degraded and flood potential is increased due to sediment delivery and subsequent deposit in stream channels. Water storage capability at the reservoirs downstream of Cheyenne could also be reduced due to sedimentation.

The long-term impact is expected to be negligible since most effects are short-term during construction. Long-term effects are not expected to be significant.

3.6.1.3.2.2 Other Areas

Site. At the LFs, erosion is estimated to increase by 16 percent. This effect would be reduced by the assumed mitigation measures relating to project construction. The impact is therefore estimated to be low and not significant. For road upgrading, erosion increases would be 31 percent within 1,000 feet of the roadway. Assumed mitigations would result in low and not significant impacts.

For cable installation, estimated erosion increase would be 23 percent for the area within 1,000 feet of the cable. Again, assumed mitigations would reduce this and control erosion at the site. The impact is therefore again estimated to be low and not significant. The difference is minor when compared to overall expected baseline erosion; therefore, the impacts are low and not significant because assumed mitigations and good construction practices will minimize erosion and protect the surface water from sediment delivery. Selection of alternative cable routes would change the location at which impacts occur, and the total regional effect, but would not alter the level or significance of impacts at the site level.

Long-term impacts are expected to be negligible and not significant.

Local. At the local level, construction at LFs would increase erosion by 0.15 percent. With assumed mitigation measures, this would be reduced and controlled at the site. Therefore, the local effect is estimated to be negligible and not significant. Similarly, local level, impacts of cable installation and road upgrading are expected to be negligible and not significant.

Regional., At the regional level, total estimated erosion due to the project is expected to increase erosion by up to 1.4 percent, depending on the watershed. With assumed mitigation, erosion would be reduced and controlled at the site. The regional impact is therefore estimated to be negligible and not significant. Long-term impacts are also negligible.

The selection of cable alternatives will affect the estimated erosion within the watershed through its relationship to total length. Based on the estimation method, the shortest routing minimizes erosion. Additional considerations would include terrain and stream crossings which were not incorporated in the erosion estimate. On the scale involved, neither the level of impact nor its significance is expected to change among alternative routes.

3.6.1.3.3 Wastewater Discharges

3.6.1.3.3.1 Crow Creek Watershed

Site. The wastewater from F.E. Warren AFB will continue to be discharged to the CBPU system. Since no discharge of wastewater is expected from the site the short-term impact is negligible. Since the short and long term have equivalent operational requirements, the long-term impacts are the same as the short term (negligible). The increase in base operations and personnel resulting from the project is considered in the local impact. Both long and short-term impacts are considered not significant since maintaining effluent

levels at legal levels as standard procedures would protect public health and safety. No changes from baseline conditions are expected for any of the road alternatives regarding wastewater discharges. Thus, analysis of alternatives is not applicable for wastewater discharges.

Local. The wastewater load on Crow Creek is expected to increase to 10,700 acre-ft/yr from the baseline flow of 10,300 acre-ft/yr, an increase of 4.0 percent in 1987. Both TSS and BOD loadings are also expected to increase by this percentage to 437 T/yr for both pollutants. This impact is rated as moderate since more effective operation of existing facilities and additions of minor new ones, such as equalization basins, would be required to meet the short-term induced wastewater demand. (See the Utilities EPTR for additional discussion concerning the Cheyenne area sewage facilities.) In the long term, the induced population causes an increased sewage flow of 135 acre-ft/yr beginning in 1991. This increase of 1.2 percent will become smaller as the baseline population of Cheyenne increases. This impact is rated as low since better operation of facilities should easily maintain treatment efficiencies at acceptable levels. The long and short-term impacts are considered not significant on a local level for the same reasons as before, i.e., standard mitigations prevent public health risks and ensure that no laws are violated.

Regional. Because the city of Cheyenne wastewater discharges are the most important factor controlling water quality in Crow Creek, the regional impacts are related to the local impacts in level and significance.

Because no facilities would be required, the impact is rated low, and since the change would not impact public health, is rated not significant.

3.6.1.3.3.2 Other Areas

Site. The waste discharges from the LCFs and LFs will continue as they have since Minuteman deployment. The construction crews will use portable toilet facilities when onsite or possibly the facilities at the LCFs. Since there is a possibility of slight but measurable increases in wastewater discharges this impact is rated low. In the long term no changes are expected from the situation as it exists with the Minuteman system, so impacts are negligible. All site impacts are considered not significant since public health is not endangered. There will be no wastewater discharges resulting from any cable route; therefore this element is not applicable.

Local. The cities and towns in the other areas will experience slight increases in population during construction. During the peak year of 1989 in Kimball, a 34 acre-ft/yr increase in flow will occur. The loadings of TSS and BOD will be increased by 1.4 T/yr. This represents a 9.3-percent increase over baseline. For Wheatland in 1986, flow and loads would increase by about 8.8 percent over baseline. For all other facilities that discharge the peak year flow and loads show no more than a 2-percent increase over baseline conditions. This will require some refined operations and additional control measures (nonstructural) to maintain surface water quality condition. The short-term, local impact is therefore low and not significant. In the long term, no cities other than Cheyenne show increased population, therefore the impacts are negligible and not significant.

Regional. The short and long-term impacts for all other areas in the ACSs are zero because no changes will occur. Therefore, regional impacts are negligible. Since no violation of law or danger to public health occur there can be no significance at this level.

3.6.1.3.4 Streamflow

3.6.1.3.4.1 Crow Creek Watershed

Site. Flow in Crow Creek may be reduced due to increased wellfield diversion or usage upstream. The effect on streamflow is not readily measurable, and no mitigation is expected. This impact is therefore considered negligible and not significant in both the short and long term. The disturbed area from road construction and changes to bridges could result in slightly altered streamflow regime. Since this impact is very small and similar for all alternatives it is considered negligible. The impacts are not significant because no safety or health concerns are generated, no laws violated, or water supplies reduced. The preproject hydrologic regime remains essentially unchanged.

Local. Flow in Crow Creek may be reduced due to upstream wellfield usage, and increased downstream of Cheyenne due to increased wastewater discharge. The effect on streamflow and water quality is not readily measurable, and no mitigation is expected. This impact is therefore considered negligible and not significant, in both the short and long term.

Regional. Flow in Crow Creek may be reduced due to upstream wellfield usage and increased due to increased wastewater discharge. The effect on streamflow and water quality is not easily measurable, and no mitigation is anticipated. This impact is therefore,considered negligible and not significant, in both the short and long term.

3.6.1.3.4.2 Other Areas

Site. Some change in flow at the sites is expected, although it is not easily measurable and is not expected to require mitigation. Some short-term degradation of water quality could occur due to erosion, although assumed mitigation should prevent or control this. The impact is considered negligible and not significant, in both the short and long term. No changes are expected due to any of the alternative cable routes. It is possible that some changes could occur at stream crossings but impacts would be negligible. Standard mitigations will render these not significant because public safety will be protected and the preproject hydrologic regime will be maintained.

Local. Some change in surface flow is expected, although it is neither easily measured nor in need of mitigation. Water quality degradation is expected to be controlled at the site. The impact is therefore expected to be negligible and not significant, both short and long term.

3.6.1.4 Groundwater Hydrology and Quality

3.6.1.4.1 Interference with Springs, Wells, and Surface Flow

3.6.1.4.1.1 Crow Creek Watershed

Site. There is not expected to be any groundwater withdrawal at F.E. Warren AFB. There will be negligible, not significant impacts in both the long and short term.

Water for road construction at F.E. Warren AFB will come from the CBPU water system and therefore may be groundwater. However, the amounts needed for any of the alternatives is small and any impacts will be negligible and not significant.

There is the possibility of meeting project-induced demands in the Cheyenne area from groundwater by: 1) additional pumpage of the Cheyenne wellfield; 2) developing new wells or leasing or purchasing water rights in the Cheyenne area in either the Arikaree or Ogallala aquifers; or 3) developing the Casper aquifer as a source of groundwater. The first two alternatives would have local impacts; the third, regional. The first alternative would produce a slight decline in water levels over baseline demand and a slight decrease in flow to Crow Creek. The impacts would be low and not significant in the short term and negligible in the long term.

Because of the restrictions imposed by state water law, the effects of alternative 2 would be minimal. The restrictions ensure that new wells or people using leased or purchased rights do not impact existing users. This includes ensuring that surface water flow is maintained to protect surface water users downstream. The impact will be negligible and not significant in the immediate future and long term.

Water requirements for the construction dispatch areas would most likely be supplied by groundwater. The amount of water required for any of the alternatives is expected to be small. Therefore, the impacts are negligible and not significant.

Regional. If water was to be withdrawn from the Casper aquifer in western Laramie County, the impacts would be both short and long term, negligible, and not significant. This is due to the restrictions on the use of groundwater imposed by state water law.

3.6.1.4.1.2 Other Areas

Site. In Wyoming, increased water use will occur for both domestic use of immigrants and construction uses. Construction uses will be considered to have site and local impacts, while the induced demands will have regional impacts.

Use of groundwater for construction purposes in Wyoming will have negligible and not significant short and long-term impacts. This is due to the fact that the regulations governing groundwater use do not allow measurable impacts without just compensation.

In Nebraska, it is much easier to develop groundwater for use and therefore there is a higher chance of interfering with existing wells, springs, or streamflow. Because the amount of water required is so small, the impacts will be low and not significant in the short term, negligible and not significant in the long term.

Groundwater will probably be used for revegetation of the cable routes. The amount of water needed will be small; therefore the impacts on springs, wells, and surfaceflow will be negligible and not significant for all of the alternative routes.

Local. Because wells supplying construction water may be located some distance away from the construction site, those demands will have local impacts as well as site impacts. The impacts are rated the same as in the previous section.

Induced demands on city water systems to supply water to inmigrants will have low, not significant impacts in the short term and negligible, not significant impacts in the long term. This is due to the fact that the municipalities will require relatively small amounts of water to meet peak induced demands and the impacts on the groundwater system will not be significant. There are no long-term water demands due to the project in any of these communities.

Regional. Negligible impacts would occur in areas outside the local area unless wells were located there. Then the impacts would be low and not significant.

3.6.1.4.2 Regional Groundwater Quality

3.6.1.4.2.1 Crow Creek Watershed

Site. Other than wastewater discharge the other major potential for groundwater degradation is due to an increase in flow percolating through the ground and the associated mineral dissolution. Because most of the water uses proposed for the project are consumptive and little return flow will occur, the potential for dissolution is minimal and the impacts, short and long term, are negligible and not significant throughout the Crow Creek watershed.

There are no expected impacts on groundwater quality due to the activities at the dispatch areas.

Local. There are no expected impacts on groundwater quality due to the activities at the dispatch areas.

Regional. There are no expected impacts on groundwater quality other than increased wastewater discharges discussed in the next section.

3.6.1.4.2.2 Other Areas

Site. There could be some impact on groundwater quality due to the use of water during revegetation. However, these impacts would probably not be measurable and therefore are rated as negligible and not significant.

Local. There are no expected impacts on groundwater quality due to the activities at the dispatch areas.

Regional. Using the rationale discussed above, the impacts on groundwater quality in the remaining ROI are negligible and not significant.

3.6.1.4.3 Wastewater Discharge Effects on Groundwater Quality

3.6.1.4.3.1 Crow Creek Watershed

Site. No wastewater discharges are expected in the area of F.E. Warren AFB. Therefore, impacts are rated as negligible and not significant, both short and long term.

There are no expected impacts on groundwater quality due to the road construction at F.E. Warren AFB since no wastewater discharge will occur.

Local. A short-term increase in wastewater discharge is expected for the Cheyenne area treatment plants due to increased wastewater associated with increased population. The increased discharges will cause a small but measurable degradation of groundwater quality, however, no violation of groundwater quality standards is anticipated. Therefore, short-term impacts are rated as low and not significant. The long-term effects are low and not significant.

It is possible that septic tank facilities may be installed at the dispatch areas to treat wastes. They would be constructed in accordance with all pertinent requirements in effect at the time. Therefore the impacts would be negligible and not significant.

Regional. No wastewater discharges would occur in the remainder of the Crow Creek watershed. However, since the City of Cheyenne is the most important factor controlling water quality in the basin, the impacts are considered similar to the local impacts, that is low and not significant in both the long and short term.

3.6.1.4.3.2 Other Areas

Site. Contamination of groundwater from increased water seepage from the lagoons at the LCFs is possible. The groundwater could show a small but measurable change in quality as nitrate or TDS concentrations may increase; therefore, the impacts would be low and not significant. Long-term impacts would be negligible and not significant.

There would be no wastewater discharges because of cable laying. Therefore, no impacts will occur related to consideration of alternatives.

Local. In the towns of Torrington and Pine Bluffs, groundwater contamination from lagoon seepage is possible. Increased flow due to inmigration to those towns could increase the seepage rate, which in turn could cause elevated nitrate or TDS levels in the area. However, increases are expected to be small and therefore the short-term impacts are rated as low and not significant, the long-term impacts as negligible and not significant because no induced population would remain once construction is complete.

Regional. Since the areas outside of the cities will have no change in groundwater quality due to discharge, the impacts in both the short and long term are negligible and not significant.

3.6.2 Aggregation of Elements, Impacts, and Significance

In order to compare alternatives containing a variety of environmental information, a systematic method of aggregating levels of impact is required. This process of aggregation can be considered a synthesis of data into a form more readily usable by the decisionmaker. Ultimately, an overall level of impact rating and significance determination needs to be assigned to water resources.

An impact aggregation or compositing procedure implies the presence and application of a value system. This is so because in an impact compositing procedure there is the inherent requirement to determine in some manner the relative importance of the various data which form the resource. In formulating the compositing methodology which follows, the National Environmental Policy Act (NEPA) was used as a basis for establishing a hierarchy of values. NEPA is specifically concerned with the following:

- The degree to which the project affects the public health and safety;
- The degree of likely controversy or uncertain risk; and
- The likelihood that federal, state, or local laws would be violated.

The method used to aggregate impacts for water resources was as follows. Aggregation of impacts took place in two steps. First, the subelement level of impact is aggregated into composite element levels of impact. Second, the four resultant levels of impacts (one for each element) are synthesized into a single level of impact rating for water resources.

The level of impact and significance for the four elements - water use and demand, constraints on water use, surface water hydrology and quality, and groundwater hydrology and quality - are defined after the environmental impacts for each subelement have been assessed and defined. The composite ranking within each element is considered equal to the highest impact ranking of any subelement within it if that impact is associated with the highest priority issues of public health and safety, potential controversy, uncertain risk, or violation of law.

The results of this aggregation process for water resources are shown in Figures 3.6-3 and 3.6-4, and discussed below.

3.6.2.1 Water Use and Demand

This element was aggregated by taking the highest impact of each subelement due to the potential controversy in any water development program.

LEGEND		ADVERSE IMPACTS	SIGNIFICANT ADVERSE IMPACTS
LEVEL OF IMPACT*	LOW	o	●
	MODERATE	○	●
	HIGH	◯	●
POTENTIAL BENEFICIAL EFFECTS			
* MEASURE OF THE AMOUNT OF ENVIRONMENTAL CHANGE			

	CROW CREEK WATERSHED						OTHER AREAS					
	SHORT TERM			LONG TERM			SHORT TERM			LONG TERM		
	SITE	LOCAL	REGIONAL	SITE	LOCAL	REGIONAL	SITE	LOCAL	REGIONAL	SITE	LOCAL	REGIONAL
Water Resources	o	●	o	o	o	o	o	o				
Water Use and Demand	o	●		o	o		◯	◯				
Legal Requirements and Constraints		o					o	o				
Surface Water Hydrology and Quality	o	●	•	o	●	o	o	o		o		
Groundwater Hydrology and Quality		o	o		o	o	o	o				

AGGREGATED IMPACT MATRIX

FIGURE NO. 3.6-3

LEGEND

LEVEL OF IMPACT Measure of the Amount of Environmental Change	Adverse Impacts	Significant Adverse Impacts
Low	o	•
Moderate	○	●
High	◯	⬤
Potential Beneficial Effects	▒	

	CABLE PATHS											ROADS			DISPATCH STATIONS		
	PA1	SB1	RB1	PA4	PD1	SB2	PB1	PA5	PA2	PA3	RB2	R1	R2	R3	PROPOSED	ALT.	NONE
Water Resources	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	
Water Use and Demand	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	o	
Legal Requirements and Constraints																	
Surface Water Hydrology and Quality												o	o	o			
Groundwater Hydrology and Quality																	

WATER RESOURCES ALTERNATIVES COMPARISON MATRIX

FIGURE NO. 3.6-4

3.6.2.2 Constraints on Water Use

This element was aggregated by taking the highest impact of each subelement due to the requirement that project facilities will not violate the law and raise potential controversy in water rights acquisition.

3.6.2.3 Surface Water Hydrology and Quality

This element was aggregated to the highest level of significant impact followed by the highest level of impact. Wastewater discharges have a higher level of impact than erosion and sedimentation on the regional level, but do not violate laws where unmitigated erosion could be controversial.

3.6.2.4 Groundwater Hydrology and Quality

This element was aggregated to the highest level of impact due to potential controversy in groundwater development and uncertain risk of wastewater discharges.

3.6.2.5 Water Resources

Aggregation of the four water resource elements to an overall level of impact and significance rating for water resources involved weighing each element (equally) to calculate level of impact and making an overall professional judgment on the significance of water resource changes due to the project.

3.7 Mitigation Measures

Certain mitigations are part of standard Air Force construction practices or policy. For potential water resource impacts these include:

- Compensation to current water users who may be impacted during project construction;
- Minimization of site disturbance and employment of proper revegetation techniques to reduce erosion potential;
- Construction of stormwater detention and erosion control facilities to control increased surface runoff impacts; and
- Development of water supply and waste disposal facilities for project-related facilities consistent with legal requirements.

The implementation of these standard mitigation measures is assumed in evaluating the level and significance of impacts on the water resource system.

3.7.1 Water Use and Demand

Mitigation measures in the area of water use and water demand fall into one of two general categories: demand reduction or supply augmentation. All such mitigations require some action with an associated cost. Overall, there may be a long-term cost or a long-term benefit associated with the mitigation. This section will describe a number of possible mitigation measures; detailed cost estimates are generally not available.

3.7.1.1 Demand Reduction Measures

3.7.1.1.1 Cheyenne Urban Area

Reduction of water demands in Cheyenne, while not an actual mitigation of project impacts, would have the effect of making more water available for project-induced growth. The City government is currently in the process of implementing some of the recommendations of the April 1982 Proposed Water Conservation Plan, including a 3-year phasing in of a uniform rate schedule. Actual demand reduction expected from this measure has not yet been projected; however, the final environmental impact statement for the Cheyenne Stage II Project estimated savings of 1,055 acre-ft/yr (USFS 1981). Recommendations of the Plan included improved metering, more efficient use of water for irrigation, use of water-saving appliances, and public education. If these measures resulted in a 5 to 10-percent reduction in water use in the Cheyenne Urban Area, a savings of about 700 to 1,400 acre-ft/yr would result. This is in line with water savings achieved in other areas implementing water conservation. Demand reductions as a result of successful water conservation efforts have not been included in baseline water demand projections.

An additional area of possible demand reduction is the reuse of treated wastewater effluent for irrigation of parks or golf courses, or for industrial purposes in the urban area. Because of downstream water rights the City must continue to discharge at current levels of approximately 6.8 mgd; however, effluent from future growth has not been appropriated and could possibly be reused. In 1987, the year of peak induced demand with project implementation, the amount of effluent that could be reused while maintaining downstream rights would be approximately 0.56 mgd (assuming reuse for irrigation at F.E. Warren AFB) or about 630 acre-ft. Without wastewater reuse at F.E. Warren AFB about 930 acre-ft would be available for reuse within Cheyenne. While reuse of wastewater for irrigation may have limited application in Cheyenne at this time, it may become more attractive from an economic standpoint as water rates increase. A preliminary analysis indicates that it may be cost effective for the Husky Oil Refinery to reuse effluent from the Crow Creek wastewater treatment plant. Additional investigations into this means of reducing water demand may be warranted.

3.7.1.1.2 F.E. Warren AFB

Currently, F.E. Warren AFB uses 225 acre-ft/yr of water purchased from the City of Cheyenne for irrigating the golf course and parade grounds on the base. The possibility of using treated wastewater effluent for this purpose has been discussed (Baker, Sweeney and Associates, 1981). With an increase in water use in the Cheyenne Urban Area this plan could be implemented. The plan consists of constructing a wastewater treatment lagoon and land application system to treat a portion of the base's wastewater. The project would reduce the projected demand on Cheyenne's water supply by about 225 acre-ft/yr, assuming no increase in irrigation water use. This reduction would occur during the summer months and would also reduce the peak demands on Cheyenne's system, thereby reducing groundwater pumpage. In addition, the amount of wastewater to be treated by Cheyenne's treatment plants would be reduced by approximately 300 acre-ft/yr. Implementation of this reuse plan would not only decrease water demands, wastewater flows, and nutrient loadings to Crow Creek, but would also result in a net savings to the Air Force over the life of the

irrigation system by reducing water and wastewater charges. The estimated cost of this system in 1982 was $1.1 million. At current irrigation water use rates this would reduce the annual water and wastewater charges by about $106,000 at the water and sewer rates projected for 1985.

3.7.1.1.3 Deployment Area

Most construction water uses for this project are consumptive e.g., compaction water is incorporated into the soil. Therefore, the amount of water that could be reused is limited. The one nonconsumptive construction water requirement, aggregate washing, has been developed assuming that wash water will be recycled and eventually used for dust control in the pit area. Construction water demand estimates include use of methods other than water spraying for dust control. Use of these methods reduces the possibility of adversely affecting existing water resources.

3.7.1.1.4 Other Municipalities

No water supply shortages have been identified in communities outside of Cheyenne. In general, water conservation measures such as those proposed for Cheyenne could be implemented in these communities if needed.

3.7.1.2 Supply Augmentation Measures

3.7.1.2.1 Cheyenne Urban Area

Although no project-induced shortages are projected for the construction period, increased water supplies would allow more rapid refilling of Cheyenne's water storage reservoirs, thereby increasing the overall reliability of the water supply. Mitigation measures could be implemented to increase supplies and assist in refilling reservoirs and reducing groundwater drawdowns.

One potential method of increasing supplies would be to rehabilitate or relocate some of the existing wells. The groundwater model developed by the Earth Technology Corporation (1983b) was used for a monthly simulation of a modified pumping pattern with a baseline annual pumpage of 2,425 acre-ft reflecting a shift from 2 wells in the Happy Jack field to 2 wells in the Bell wellfield. This shift in the pumping resulted in a more evenly distributed drawdown pattern. The area showing the maximum simulated drawdowns (21 feet) with the current pumping distribution shows a drawdown of 12 feet with the modified pattern. Also, the modified pattern does not result in any sizeable increase in drawdown anywhere in the wellfields as compared to the current pumping pattern. This suggests that increased quantities of water could be supplied from the municipal wellfields with minimal impact to the system. Results of testing modified pumping patterns indicate that patterns could be developed with existing wells which would better distribute water-level declines associated with baseline or project-induced withdrawals. A limitation to implementing modified pumping patterns is the condition of a number of the existing wells, many of which require rehabilitation to achieve maximum yield and efficiency.

The potential exists for the augmentation of existing surface water supply capability through snow management methods. These methods are typically the

construction of barriers to snow transport, such as fences, to increase accumulation and reduce sublimation of windblown snow. In the Crow Creek watershed, basins typified by rolling grasslands are conducive to snow management because losses to sublimation are high, while basins incorporating forestation and/or rugged terrain are not. The Middle Fork of Crow Creek above Granite Springs Reservoir and the North Fork above the North Crow Creek Reservoir are not considered conducive to augmentation.

A study of the Middle and South Forks of Crow Creek has been conducted to evaluate snow management potential for improving watershed yield (Tabler 1971). The South Fork was noted to have a runoff rate one-third lower than that of the Middle Fork. This difference was attributed to differences in snow capture due to barriers of topography and vegetation in the two basins. The Middle Fork was noted to contain more natural barriers, and 80 percent of runoff was attributed to snowmelt, compared to only 73 percent in the South Fork Basin.

Experimentation with snow fences in several small basins of 100 acres near Pole Mountain showed a 70-percent increase in snowpack water equivalent. Of this amount, roughly one-third would be expected to augment surface flows, with the remainder lost to evaporation and soil infiltration (Sturges and Tabler 1981). For total basin yield, this would correspond to a 17-percent increase in flow (one-third of 70 percent increase in snowfall capture times 73 percent of yield attributable to snowmelt).

On a larger scale, a lower increase in yield could be expected due to the lack of applicability throughout the watershed and possibly greater losses to infiltration. An assumed increase of 5 percent is used in this analysis to evaluate augmentation potential.

For basins in which augmentation is considered feasible, a total of 3,926 acre-ft of water are available for water supply use. These totals are: North Fork below diversion and Brush Creek - 1,970 acre-ft; Middle Fork below Granite Springs Reservoir - 911 acre-ft; and South Fork - 1,045 acre-ft. The potential augmentation for these basins totals 196 acre-ft of yield. Of these areas only the Middle Fork is currently used for water supply, and this basin has a potential augmentation of 45 acre-ft. If all basins could be augmented 17 percent, comparable to the experimental results, a maximum potential of 667 acre-ft of additional yield could be achieved.

Based on experimental results and field experience, cost for such a system is roughly comparable with developing new water sources (USFS). Cost trends are improving the cost-effectiveness of augmentation measures. Design of the system should be based on cost-effectiveness of snow capture and fences should be located where snow would collect in or near drainage channels in order to minimize infiltration losses.

Improvements to the Brush Creek Diversion would allow CBPU to use this facility to obtain additional surface water supplies. The current system valving controls both the infiltration and surface diversions. By providing a separate valve for each it would be possible to use the infiltration water even when the surface diversion would be shut off because of turbidity problems. The possibility of constructing a small holding reservoir to allow settling of suspended solids above the surface water diversion could also be

investigated. If turbidity problems could be reduced in this manner, or by operational modifications at the water treatment facility, the yield of the North Crow Creek watershed could be increased appreciably.

In addition, the South Crow Diversion is not currently used because of turbidity problems. The potential yield of this diversion is 1,100 acre-ft/yr. If structural improvements of the diversion dam or reservoir could be made to improve settling prior to diversion, then CBPU could resume use of this diversion. The potential increase in water yield warrants some investigation into this matter. It is also possible that operational changes at the water treatment plant or some form of pretreatment could be implemented to decrease problems associated with turbidity. The possibility of using Round Top Lake to reduce turbidity in surface water supplies prior to treatment could also be studied.

An additional mitigation to increase available water supplies is the development of new sources of groundwater. The Casper Formation on the eastern flank of the Laramie Range and the Ogallala/Arikaree aquifers northeast of Cheyenne have been discussed in this respect. However, because of the length of time required for exploration, water rights acquisition, and construction it is not likely that these sources would be available to meet the project-induced demands. In addition, the cost of these alternatives would be quite high especially considering the magnitude of the induced demands (maximum of 571 acre-ft/yr).

Lease or purchase of existing groundwater rights within the Crow Creek watershed could be used to provide additional water to the Cheyenne Urban Area to mitigate the project-induced demands. Further investigation would be required to identify those rights which could be acquired and to determine in which locations the supply could be most efficiently transmitted to the Cheyenne Urban Area.

3.7.1.2.2 F.E. Warren AFB

Because F.E. Warren AFB receives its entire water supply from the CBPU, the supply augmentation measures for Cheyenne previously discussed also apply indirectly to the base. If wastewater reuse for irrigation at the base does not prove to be implementable, then it is possible that the Air Force could acquire groundwater rights for irrigation and reduce their purchase of water from Cheyenne. While this would probably not be as cost-effective as wastewater reuse, it would result in additional water supplies to meet project-induced demands. In acquiring water rights, either temporary or permanent, the Air Force would comply with all regulations of the Wyoming State Engineer. This would ensure that existing groundwater users would not be impacted.

3.7.1.2.3 Deployment Area

Wells in the DA adequately serve current and future needs at LCFs with or without project development. No supply augmentation is necessary to meet LCF demands. No mitigations other than the standard assumed measures are required for impacts in the DA.

3.7.1.2.4 Other Municipalities

Only negligible impacts in municipalities other than Cheyenne have been identified due to project development, therefore no water supply augmentation measures are presented as mitigation measures.

3.7.2 Constraints on Water Use

The WDEQ and the NDEC have each established procedures requiring permits for construction of sedimentation ponds, public water supplies (greater than 20 service connections from point of supply) and for facilities for treatment and disposal of wastewater. Compliance with the regulations is standard Air Force procedure.

Water acquisition efforts for the project would follow the requirements of Wyoming and Nebraska water laws. Mitigations would include payment for any detectable impact on existing water rights determined as a result of the required monitoring programs, reduction of well discharges, or relocation of wells that show signs of impacting existing water right holders.

The existing wells located at the LCFs in Wyoming have not been permitted by the Wyoming State Engineer and are therefore in violation of current Wyoming water laws. Applications for a permit for existing miscellaneous uses should be filed.

3.7.3 Surface Water Hydrology and Quality

3.7.3.1 Stormwater Detention

Stormwater detention facilities would be constructed to reduce projected flood and erosion impacts on the Cheyenne Urban Area, the Crow Creek watershed, and the area around silos, roadways, and control cable installations. Stormwater detention facilities limit peak surface runoff by restricting outlet releases and accumulating the difference between inflows and regulated outflows. The accumulated stormwater is then released after the peak flood has passed. The City of Cheyenne has developed and requires onsite stormwater detention to limit peak developed surface runoff to historic rates. All construction in Cheyenne will follow these regulations. All construction not under the control of Cheyenne regulations will be performed according to all applicable local, state, and federal regulations in force at the time of construction.

The WDEQ has adopted "Sedimentation Pond Review Criteria" and requires permits prior to construction of such facilities. By adhering to these criteria at all construction sites, impacts due to runoff and erosion would be mitigated.

3.7.3.2 Erosion Control and Revegetation

Development results in increased surface runoff, disturbs the soil surface, and increases sheet or rill erosion. Consequently, the sediment transport capacity of the major drainageway and watershed sediment yield would be increased.

Disturbance of a relatively small portion of the surface area of a watershed can dramatically increase watershed sediment production. This increase can result in measurable changes in erosion and deposition along the main stem downstream of the disturbed area. Sediment production is thought to first increase dramatically following disturbance of the soil surface and then rapidly decline as the soil surface stabilizes during the first years following construction. Temporary silt fences and wind rowing can significantly mitigate accelerated sediment production during the first few years following construction.

Revegetation of disturbed areas can eventually restore the erosion resistant surface of the watershed mitigating increased sheet or rill erosion associated with disturbance of the soil surface and is the long-term mitigation approach to be followed.

Stormwater detention basins also reduce the sediment transport capacity of surface storm runoff causing excess sediment to drop out and deposit within the basin preventing deposition further downstream. Caution should be exercised to assure that stormwater detention/sediment control basins do not reduce sediment loads below present values or erosion will occur along the main stem downstream of the control facility. The use of stormwater detention facilities to reduce flood impacts also would serve to reduce erosion impacts particularly during project construction before revegetation is complete. Various other erosion control practices (mulching, etc.) should also be adopted to reduce erosion potential during this period.

3.7.3.3 Wastewater Discharges

Increased wastewater discharges from the Cheyenne Urban Area would be an important project effect on surface water quality. The increase in wastewater loads from other facilities within the ROI could cause changes to surface water quality. Wastewater treatment and discharge from any facility will be required to comply with National Pollutant Discharge Elimination System permit limits set forth by the WDEQ and NDEC. Compliance with permit regulations would mitigate any impacts resulting from such discharges.

3.7.3.3.1 Cheyenne Urban Area

The three existing Cheyenne area sewage treatment facilities have been investigated in a recent study (Banner Associates 1982). This 201 Facilities Plan indicates that while the facilities would not be overloaded they will still be unable to provide sufficient wastewater treatment for the baseline growth of Cheyenne. If the facilities are not enlarged and upgraded, treatment efficiencies will continue to decline and effluent limitations will not be met. The Cheyenne 201 Facilities Plan (Banner Associates 1982) discusses the measures required to provide adequate treatment for Cheyenne in the future. The measures discussed in the document would serve to mitigate impacts if the project is initiated.

The Facilities Plan recommends that the South Cheyenne Water and Sewer District (SCW&SD) treatment plant be abandoned and replaced with a 2.5 mgd pumping station. The flow would then be pumped to the Crow Creek or Dry Creek facilities which are operated by the CBPU. The Crow Creek facility is to be upgraded so that at its design capacity (4.0 mgd) it can consistently meet

secondary effluent limits. The Dry Creek facility is to be expanded to handle 7.0 mgd and various treatment processes are to be upgraded so that the plant consistently meets secondary effluent limits.

The recommendations of the Facilities Plan could be implemented at an earlier time, thus providing sufficient capacity to handle the expected flow of 7.7 mgd by 1987. If all Facility Plan recommendations could be implemented by 1987, the Cheyenne sewage treatment facility would have sufficient capacity to treat the wastewater flow generated by the total population of the Cheyenne area with project deployment. (Further details are contained in the Utilities EPTR.)

Reuse of the wastewater that would be discharged to Crow Creek could also reduce water quality impacts of discharges.

Another possible mitigation measure with similar legal questions is to design and build a zero discharge facility to treat a portion of the wastewater flow. This would prevent additional pollutant loadings to Crow Creek that would result from the increased population. The problem again is the question as to how much water could be treated in the zero discharge facility and the quantity that must be discharged to satisfy downstream junior appropriations.

Since Crow Creek is a Class IV stream, treatment beyond secondary is not required. However, a possible mitigation measure could be to increase the level of treatment to offset the additional loading on Crow Creek.

3.7.3.3.2 Other Areas

The DA will continue to generate wastewater from the LCFs and LFs. The LCFs currently treat their sanitary sewage in systems of septic tanks followed by a lagoon. The LFs maintain a sump that is periodically pumped out to remove moisture that may accumulate in each silo.

Each of these facilities will continue to handle all wastewaters generated in an environmentally acceptable manner and meet applicable discharge regulations. This water could be reused; however, the minimal amount available at each LCF or LF will not be useful for irrigation or revegetation. Water use at the LCFs averages only 1,000 gallons per day so wastewater production will be less than this. No good data on the quantity of water removed from each LF are available.

For individual areas outside Cheyenne which may be served by onsite or cluster systems, the most important mitigation measure is proper design and operation of the septic tank system(s). The applicable health and zoning requirements for septic systems should be followed to ensure that these wastewater discharges are handled in the most environmentally safe and efficient manner possible.

For communities other than Cheyenne, the treatment of sanitary wastewater will continue at or above current levels. The communities of Pine Bluffs, Torrington, and Wheatland, Wyoming and Kimball, Gering, and Scottsbluff, Nebraska will have increased populations due to the project. All other communities in the ROI will not be affected by the project and thus no mitigation measures will be required to prevent deterioration of surface water

quality. Some of these communities may upgrade and/or enlarge their sewage treatment facilities; however, any of this work will be done as routine maintenance, replacement, planned upgrade of old systems that are not functioning properly, or replacement of individual septic systems and will not be required because of the project.

Of the eight towns affected by the project, Pine Bluffs, Albin, Chugwater, and Torrington utilize zero discharge facilities, thus no additional mitigation measures to protect surface water will be required. Wheatland has in the past treated sewage from a population of approximately 9,000 persons during construction of the Laramie River Power Station. This indicates a capacity to handle more people than expected from the project and any growth that might occur although none is expected. No mitigation measures will be required in these Wyoming communities.

The three communities in Nebraska that may require mitigations to further protect surface water quality may be able to use measures similar to those discussed for the Cheyenne area. Upgrading, enlarging, water reuse, conservation, or combinations of these may be used. Since these communities are required to meet their discharge limits in the future and the expected population changes due to the Proposed Action are small, it is not anticipated that their effluent discharges will cause surface water quality degradation. No mitigations other than those necessary for normal population changes and sewer system alterations would be necessary.

3.7.3.4 Monitoring and Modeling

Monitoring of sediment and erosional tendencies is accomplished by observation and inspection and the establishment of permanent channel cross section locations that can be periodically resurveyed to quantify the amount of sediment deposition or erosion that has taken place. Monitoring stations could be established in drainageways that will be affected by construction. These stations would serve as observation points to determine if sediment is being deposited or removed. The stations would be established when sediment control structures are built and would be observed during construction and operation. Stations would be located as needed at each silo, around DAR upgrade areas, cable routes, and in F.E. Warren AFB if a potential erosion problem exists. Because of the site-specific nature of erosion and siltation changes, each LF, DAR, cable route, and area on F.E. Warren AFB will be evaluated on an individual basis.

There will be no scheduled program for primary data acquisition at these sites. The primary function would be to qualitatively assess the success of erosion control measures. When construction is completed, the sediment control facilities will be removed (if any were used) and the drainageway returned to its original condition.

The development in the City of Cheyenne will occur in basins that could affect Crow Creek, as described in Section 3.5.3.1.1. The Crow Creek/Cheyenne Urban Area could benefit from employing a similar qualitative monitoring system whereby periodic inspection of select stream sections would be done to determine if erosion or sediment deposition is taking place.

Physical and chemical water quality monitoring could be accomplished by using

existing monitoring stations that are operated by USGS. These stations and their descriptions have been noted in sections on water quality in Section 2.6.3. Also the monitoring program done by the Air Force on F.E. Warren AFB will be continued. This type of program requires a well managed data handling and storage system. With this system in place, results of chemical analyses of the samples taken from streams in Crow Creek may be compared to results from the sampling program prior to initiation of the project to evaluate any change in the quality of surface waters.

Because of the existing water quality conditions in Crow Creek and the very small changes expected in the other watersheds, extensive modeling of water quality using sophisticated modeling techniques will not be required.

A program of monitoring and modeling is being developed in conjunction with the water development plans. The Air Force will comply with the requirements of both states in developing the water supply for the project, including all monitoring requirements. When the water supply program is completely identified, the monitoring and modeling program will be more fully defined.

Finally, because of the strong relationship between surface water and groundwater, any programs for monitoring and modeling these two resources should be closely coordinated. See Section 3.7.4.4 for a discussion of potential groundwater programs.

No other water quality impacts are expected on Lodgepole, Horse, Pumpkin, or Chugwater creeks or other watersheds located in the ROI and no monitoring would be required.

3.7.4 Groundwater Hydrology and Quality

3.7.4.1 Well Spacing and Construction

The impact of project wells, if developed, could be mitigated by proper well spacing and construction. Any wells constructed as a result of the project should fully penetrate the aquifer. The method of drilling will require test holes to find the deepest area of the aquifer and highest permeabilities. The production wells should be drilled where the aquifer is deepest and has the maximum permeabilities. The lower 50 percent of each well should be screened. The development of the well should be such that the well efficiency will be at least 80 percent.

The wells should be located such that they are not closer than 1,000 feet to the nearest flowing stream for wells producing 100 gpm. This would ensure minimal depletion of flowing streams and maximize the infiltration of precipitation. The spacing between wells should be at least twice the above referenced distances to minimize mutual interference between wells.

3.7.4.2 Aquifer Recharge and Storage

The production of water from any of the post-Cretaceous or Casper aquifers would not effect recharge to the aquifer if the above spacings are followed. The total amount of water in storage in any given aquifer would be temporarily reduced as a result of the additional pumping required by the project. After the cessation of pumping the aquifer storage would return to the prepumping

levels as a result of precipitation infiltration.

As a result of the reduced storage, the discharge to streams and groundwater outflow will be decreased somewhat. The reduced discharge to the streams, in that they are over appropriated, may have to be mitigated by the purchase or lease of senior surface water rights. There is not expected to be any reduction of water pumped from the aquifers for irrigation, municipal, domestic, or industrial uses. The evapotranspiration of groundwater would not be significantly affected.

3.7.4.3 Wastewater Management

The recharge to the unconfined aquifers, as a result of the increased discharge of municipal wastewater effluent to Crow Creek, would increase as a result of the project. This recharge would help offset the reduction of streamflow and groundwater outflow outlined above. There may be some reduction of groundwater quality as a result of this recharge.

Increased recharge to the groundwater may occur as the result of increased wastewater flow to the no discharge lagoons in Pine Bluffs, Torrington, and at the LCFs. While this increase in recharge will help replace construction water pumped from the ground, it may also affect the quality of the groundwater. One method of mitigating this impact would be to line the lagoons with clay or a synthetic liner. This would prevent seepage into the ground and all flow to the lagoons would eventually be evaporated. Some lagoons may have to be enlarged to provide sufficient storage if this is done.

3.7.4.4 Monitoring and Modeling

The numerical groundwater model for the Crow Creek watershed, developed by the Earth Technology Corporation (1983b), has been run for conditions simulating operation of the Cheyenne wellfields. The results of this modeling investigation are presented in Section 3.5.4.2.1.1. A scenario of reducing pumpage in the western wells and increasing pumping of the northeast wells was modeled. The results showed reductions in drawdown and a more evenly distributed drawdown. The modified pumping pattern could be implemented as a mitigating measure to lessen any impacts if the wellfields are pumped at a higher rate.

In order to predict future impacts in the Crow Creek watershed and determine appropriate mitigations, groundwater modeling would be necessary. The methods used to determine project effects included deductive reasoning and analytical models. These models are simplifications of complex physical situations and are not as accurate as a finite difference or a finite element model. Therefore, a computer model may be the most appropriate method to further define these effects. Additional monitoring may be necessary to augment the data base such that application of the models will yield valid results. Monitoring and modeling of groundwater withdrawal impacts in the DA and in the Cheyenne wellfields may be part of the mitigation program for the project.

In the Deployment Area very little monitoring or modeling would be necessary except to meet State requirements for any new project-related well.

3.7.5 Summary

Potential mitigation measures that will be considered are identified below. One, some, or all of the mitigation measures may ultimately be selected. Each measure identifies the party responsible to implement, but not necessarily to pay for, the measure.

- Implementation of water conservation measures or wastewater reuse projects in the Cheyenne Urban Area or on F.E. Warren AFB to reduce raw water demand on F.E. Warren AFB by about 225 acre-ft/yr and nearly equals long-term project-related demand increases. It is difficult to determine exact reduction amounts in Cheyenne due to water conservation, but it would at least maintain per capita use at present levels. This conservation program has been initiated in Cheyenne by restructuring the water rate charge system and beginning a public education program. Water reuse projects would require construction of some facilities, and therefore design should (if selected) begin in early 1984 so that these facilities are in place at the time of peak water demand. The responsible agency for implementing these measures would be the Air Force and the Cheyenne Board of Public Utilities.

- Operation and/or modification to raw water collection systems in the Crow Creek watershed to increase use of the available supply. Depending on how extensive a modification to the system is selected, this mitigation could increase available supply by up to approximately 2,000 acre-ft/yr. Increasing supply would have a long-term benefit to the Cheyenne Urban Area. This measure should be implemented in early 1984 if major structures (e.g., new settling reservoirs) are required or in mid-1984 if minor facilities (e.g., new valve for Brush Creek Diversion) are selected. The responsible agency for this measure is the Cheyenne Board of Public Utilities.

- Lease or purchase of water rights in the Crow Creek watershed to provide additional water to the Cheyenne Urban Area. This measure would require legal investigation of the water rights in question, negotiation with present owner(s) and application for change of use or location with the state engineer. In order to acquire enough water for the project, the available rights must be researched to determine which rights could be leased or purchased. This measure could be used to meet the entire project demand without major change to the area's hydrology. Since this process could require more than 1 year to change use or location, this measure should be implemented in early 1984 to meet demands in Cheyenne. The agency responsible for this measure is the Cheyenne Board of Public Utilities.

- Use of stormwater detention facilities and/or erosion control including revegetation for all nonproject-related construction projects in the Cheyenne Urban Area. These would be in addition to the assumed mitigations implemented by the Air Force. Implementing this measure could reduce expected peak flows in the Cheyenne Urban Area to less than or equal to historic rates. Erosion control mitigation will reduce projected erosion quantities to at or below existing levels. If both were implemented, there would be little

long or short-term impact on surface water hydrology in the Cheyenne area. This measure should be implemented early to maximize mitigation effects. The agencies responsible for this mitigation measure would be the Cheyenne and Laramie County Public Works departments and Air Force.

- Early implementation of plans for upgrading of wastewater treatment facilities in the Cheyenne Urban Area to reduce discharge impacts. Planning for this measure has been initiated with the facilities planning effort (Banner 1982). In order to implement the recommendations in the plan, design should be initiated in the first quarter of 1984. Even with an accelerated schedule of design and construction it is likely that the peak population impact would have passed before the facilities are complete. However, water quality would be improved from baseline conditions. If this measure were used, the sewage treatment capacity in the Cheyenne area would be 11 million gallons per day which would easily serve the expected population. The agency responsible for implementing this mea;sure is the Cheyenne Board of Public Utilities.

- Siting of new wells, if any, in the Deployment Area at least 2,000 feet from existing streams and springs to minimize impacts to spring or streamflow. This mitigation measure would be implemented as new wells are needed in the Deployment Area and construction activities progress to different sites throughout the Deployment Area. This would be effective in minimizing water use impacts in the Deployment area. The responsible agency for implementing this measure is the Air Force.

- Increase monitoring and modeling of the Cheyenne wellfield operations and raw water supply in the Crow Creek watershed to better define production capacity. Groundwater model simulations were done for the Crow Creek wellfields and will continue to be refined into 1984. A better understanding of the impact of operations, rehabilitation or relocation of wells, and monitoring would allow the wellfields to be used to meet all project-related water demands in Cheyenne with only low or negligible effects on water resources. It would provide long-term benefit to the Cheyenne area. The responsible agency for implementing this measure is the Air Force in conjunction with the Cheyenne Board of Public Utilities.

3.8 Unavoidable Adverse Impacts

Implementation of the Proposed Action would result in consumptive use of about 4,000 acre-ft of water during the 1984 through 1990 time period. The water resource system response to this use would generally impact the water delivery system to, and wastewater discharges from, the Cheyenne Urban Area, where approximately 2,730 acre-ft of use would occur. Construction would permanently alter drainage characteristics in several drainage areas. Implementation of the Proposed Action would increase long-term water demands by about 250 acre-ft/yr in the Cheyenne Urban Area. There would be no other long-term increase in water use as a result of the project.

3.9 Irreversible and Irretrievable Resource Commitments

The two irreversible and irretrievable commitments from the water resource system are a consumptive use of water and changes to drainage patterns. During construction of the project there will be 4,000 acre-ft of water consumptively used. During the operational phase of the project 250 acre-ft/yr will be required. Drainageway characteristics altered by development will be permanently changed. While drainageways can be restored it is not possible to return them to their exact preproject condition.

3.10 Relationship Between Local Short-Term Use of Man's Environment and Maintenance and Enhancement of Long-Term Productivity

During construction of the project 4,000 acre-ft of water will be consumptively used and will not be available for other uses (e.g., agricultural, municipal, etc.). In the long term, an additional 250 acre-ft/yr of water will be needed for the project. This amount of water would reduce available supply that could potentially be used for other purposes.

Permanent alteration of drainage patterns could potentially alter riparian habitats, change flooding patterns, and could possibly increase the productivity of some areas by reducing impacts of erosion and siltation downstream that would have existed without stormwater detention facilities, erosion control measures, and channel improvements used during the project.

GLOSSARY

4.0 GLOSSARY

4.1 Terms

Acre-Foot: the volume of water that would cover 1 acre to a depth of 1 foot.

Alluvium: sediments deposited by a stream.

Aquifer: the water-bearing portion of subsurface earth materials that presently yields or is capable of yielding useful quantities of water to wells.

Baseline: the characterization of an area under no-project conditions.

Cenozoic: an era of geologic time extending from about 65 million years ago to the present (USGS).

Center Pivot Irrigation: an irrigation system utilizing a moving framework of sprinkler heads which is anchored at one end and is propelled in a circle, usually one-eighth mile in radius, by water pressure.

Climate: the prevalent or characteristic meteorological conditions, and their extremes, of any given location or region.

Cretaceous: the last period of the Mesozoic era, extending between 136 and 65 million years ago (USGS).

Cumulative Effects: the aggregation of project-induced effects and the effects of other concurrent projects occurring simultaneously in the project's Region of Influence (ROI). The term cumulative has also been used to denote aggregated effects over several years as against net effects in a given year.

Deployment Area (DA): the 319th and 400th Squadrons at F.E. Warren AFB where Peacekeeper missiles will replace Minuteman missiles, including road and communication routes.

Direct Effects: effects resulting solely from project implementation.

Disturbed Area: that specific land which has had its surface altered by grading, digging, or other activity related to construction.

Dry Cropland: land devoted to the production of crops without the need for irrigation.

Effect: a change in an attribute. Effects can be caused by a variety of events, including those that result from project attributes acting on the resource attribute (direct effect); those that do not result directly from the action or from the attributes of other resources acting on the attribute being studied (indirect effect); those that result from attributes of other projects or other attributes that change due to other projects (cumulative effects); and those that result from natural causes (e.g., seasonal change).

Ephemeral Stream: a stream that flows briefly only in response to precipitation in the immediate vicinity and whose channel is at all times above the water table.

Floodplain: for inland waters, the area subject to a 1 percent or greater chance of flooding in any given year (i.e., the area adjacent to a stream expected to be inundated in a 100-year flood). Executive Order 11988, Floodplain Management, places limitations on construction projects in floodplains and promulgates guidelines to ensure public health and safety both to protect against property loss and to protect natural and beneficial values of floodplains.

Formation: a sequence of similar rock layers that can be traced over a large area.

Fringe Area: that unincorporated area adjacent to Cheyenne containing residential and nonresidential uses similar in character to the incorporated portions of Cheyenne.

Habitats: places or physical areas with particular kinds of environments in which organisms live.

Hydrogeologic Unit: one or more geologic formations with similar water-bearing and transmitting characteristics.

Impact: an assessment of the meaning of changes in all attributes being studied for a given resource, an aggregation of all the effects, usually measured using a qualitative and nominally subjective technique.

Impoundment: a man-made area for the purpose of detention or retention of surface water.

Indirect Effects: effects resulting from the attributes of other resources acting on the attribute being studied. For example, direct project employees will spend some of their income locally. As a result, local industries will tend to hire more workers as they expand in response to the increased demand. This additional employment is termed an "indirect effect."

Infrastructure: the system of public utility lines, communication facility networks, and roadways which connect all the structures and facilities in a given locale.

Inmigrants: all people relocating into a defined geographic area usually calculated on an annual basis.

Intercontinental Ballistic Missile (ICBM): a large, land-based missile capable of accurate delivery over intercontinental ranges (usually greater than 5,000 miles).

Intermittent Stream: a stream that does not have continuous flow during all periods of the year.

Irrigated Cropland: land devoted to the production of crops which require and benefit from periodic irrigation.

Landowner: a person or entity indicated as the owner of property on the various ownership maps maintained by the Office of the County Assessor.

Local: for the Crow Creek watershed Area of Concentrated Study, local is the Cheyenne Urban Area; for the other Areas of Concentrated Study, it is 2 miles from silos, towns, and roads.

Mean: a value that is computed by dividing the sum of a set of terms by the number of terms; average.

Mesozoic: an era of geologic time extending from about 195 million to 65 million years ago (U.S. Geological Survey).

Milligram: one-thousandth of a gram.

Millimeter: one-thousandth of a meter.

Mitigations: methods to reduce or eliminate adverse project impacts.

Mobile Home: a single-family dwelling unit which is transportable in one or more sections, built on a permanent chassis, and designed to be used with or without a permanent foundation. Does not include travel trailers or recreational vehicles.

Model: a mathematical formula that expresses the actions and interactions of the elements of a system in such a manner that the system may be evaluated under any given set of conditions.

Noncompliance: action contradicting a specified procedure or causing results outside specified limits.

Paleozoic: an era of geologic time extending from 570 million to 195 million years ago (USGS).

Peak Flow: the maximum discharge of a stream during a specified period of time.

Peak Year: the year in which some particular project-related effect, e.g., total water use, is greatest.

Perennial Stream: a stream that has continuous flow during all periods of the year.

Precambrian: all geologic time before the beginning of the Paleozoic era, equivalent to about 90 percent of all geologic time.

Promulgate: to make known or public the terms of a proposed law.

Quaternary: the second period of the Cenozoic era extending from 2 to 3 million years ago to the present.

Rangeland: that land devoted to the grazing and keeping of animals such as cattle, sheep, and horses.

Reclamation: the process of restoration of an area which has been disturbed.

Regional: the watershed in which activities occur.

Revegetation: regrowth or replacement of a plant community on a disturbed site. Revegetation may be assisted by site preparation, planting, and treatment, or it may occur naturally (secondary succession).

Riparian: pertaining to features on the bank of a natural watercourse.

Rural: that area outside of towns, cities, or communities; characterized by very low density housing concentrations, agricultural land uses, and general lack of most public services.

Section: a subdivision of a township that is 1 mile square.

Significance: the importance to the resource of the impact on the resource. Council on Environmental Quality (CEQ) regulations specify several tests to determine whether an action will significantly affect the quality of the human environment. While these tests apply to the entire action, they can also be used in an amended form to judge impact significance for individual resources. It is important to note that a high impact may not be significant, while a low impact may. Significance is an either/or determination: the level of impact described either is significant or is not significant. Additionally, beneficial significance must be determined at the same level as adverse significance. As specified in the CEQ regulations, significance needs to be determined for each of three geographic areas: local, regional, and national. This places the impact into context. Significance is also determined in terms of intensity.

Site: for the Crow Creek Area of Concentrated Study, site is F.E. Warren AFB; for the other Areas of Concentrated Study, it is 1,000 feet from silos and roads.

Site Specific: conditions characteristic of a geographically defined location that may vary considerably from characteristics of adjacent locations or the characteristics of a larger area within which the location in question is contained.

Sublimation: an effect similar to evaporation, whereby a substance passes directly from a solid to gaseous phase. For example, snow or ice can sublimate into the atmosphere without ever becoming a liquid.

Terrace: a flat portion of land created by the action of a stream or river before it moved to a different location.

Tertiary: the first period of the Cenozoic era extending between 65 million and 3 to 2 million years ago.

Township: a surveyed tract of land containing 36 square miles and identified relative to its relationship to defined parallels of latitude.

Unincorporated: not included within the corporate limits of a city or town.

Urban: that area within towns, cities, or communities, characterized by densities greater than one dwelling unit per acre.

Wetlands: areas defined by the prevailing vegetation and soil moisture content and consisting of vegetation typical of soils that are saturated for a major portion of the year.

Worst Case: the combination of all the worst possible effects to result potentially from the actions of a project.

201 Plan: wastewater treatment plan (facilities plan) developed pursuant to Section 201 of Public Law 92-500.

4.2 Acronyms

ACS	Area of Concentrated Study
AFB	Air Force Base
AFRCE	Air Force Regional Civil Engineer
BLM	Bureau of Land Management
BMS	Ballistic Missile Support
BOD	Biochemical Oxygen Demand
BOD_5	5-Day Biochemical Oxygen Demand
CBPU	Cheyenne Board of Public Utilities
CEQ	Council on Environmental Quality
CFR	*Code of Federal Regulations*
COE	Corps of Engineers
DA	Deployment Area
DAR	Defense Access Roads
DO	Dissolved Oxygen
DoD	Department of Defense
DOI	Department of the Interior
DOPAA	Description of Proposed Action and Alternatives
EIAP	Environmental Impact Analysis Process
EIS	Environmental Impact Statement
EPA	United States Environmental Protection Agency
EPTR	Environmental Planning Technical Report
FEIS	Final Environmental Impact Statement
FEMA	Federal Emergency Management Agency
FY	Fiscal Year
ICBM	Intercontinental Ballistic Missile
LCF	Launch Control Facility
LEIS	Legislative Environmental Impact Statement
LF	Launch Facility
LOI	Level of Impact
MCL	Maximum Contaminant Level
M-X	Missile Experimental
NCSD	University of Nebraska Conservation and Survey Division
NDEC	Nebraska Department of Environmental Control
NDH	Nebraska Department of Health
NDWR	Nebraska Department of Water Resources
NEPA	National Environmental Policy Act
NOAA	National Oceanic and Atmospheric Administration

OB	Operating Base
PP&L	Pacific Power and Light Company
ROI	Region of Influence
SCS	Soil Conservation Service
SCW&SD	South Cheyenne Water and Sewer District
SS	Suspended Solids
SWMM	Storm Water Management Model
TDS	Total Dissolved Solids
TSS	Total Suspended Solids
USAF	United States Air Force
USDA	United States Department of Agriculture
USFS	United States Forest Service
USGS	United States Geological Survey
WDEQ	Wyoming Department of Environmental Quality
WSEO	Wyoming State Engineer's Office
WWDC	Wyoming Water Development Commission
WWRC	Wyoming Water Research Center
WWTF	Wastewater Treatment Facility

4.3 Units of Measurement

acre-ft	acre foot
acre-ft/yr	acre feet per year
^{o}C	degrees Celsius
cf	cubic feet
cfs	cubic feet per second
^{o}F	degrees Fahrenheit
ft	foot or feet
ft/day	feet per day
ft/mi	feet per mile
g	grams
gpcd	gallons per capita per day
gpm	gallons per minute
gpm/ft	gallons per minute per foot
lb	pound
lb/day	pounds per day
mg	milligrams
MG	million gallons
mgd	million gallons per day
mg/l	milligrams per liter
ml	milliliter
pCi/l	picoCuries per liter
ppm	parts per million
sq ft	square foot or feet
sq ft/day	square feet per day
sq mi	square mile
T/day	tons per day
T/yr	tons per year

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5.0 REFERENCES CITED AND REVIEWED

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6.0

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APPENDIX A

APPENDIX A
ANALYTIC METHODS

Literally hundreds of textbooks and publications have been written describing analytical methods for water resource analysis. Each of the analytical methodologies mentioned in Section 2.5 of this report has been concisely summarized in this appendix. Such a summary does not allow the reader to have a detailed knowledge of the application of these methods and all of the various factors and assumptions necessary for proper application to a particular problem. The interested reader is directed to the reference publications for more detailed information.

The intent of this appendix is to describe the major features of each method. Each method described here is summarized in terms of what it does, major equations or techniques used, required input data, and output data. The limitations of the method and the justification for using the method are also presented. Finally, the references used for preparing the summary are provided.

Good general references for water resource analysis techniques available include:

1) "Areawide Assessment Procedures Manual Volumes I, II and III." U.S. Environmental Protection Agency EPA-600/9-76-014, July 1976.

2) "Proceedings of Conference on Environmental Modeling and Simulation held April 19-22, 1976, Cincinnati, Ohio" U.S. Environmental Protection Agency.

3) Groundwater Manual, U.S. Bureau of Reclamation, U.S. Department of the Interior, Water.

The methods included in this appendix are as follows:

Surface Water Hydrology and Quality

A.1 Lowham Method
A.2 Technical Release 55 Methods
A.3 Technical Release 20 Methods
A.4 HEC-1 Method
A.5 HEC-2 Method
A.6 Urban Runoff Method
A.7 SWMM
A.8 Modified Streeter-Phelps Equation

Groundwater Hydrology and Quality

A.9 U.S. Geological Survey Two-Dimensional Groundwater Model
A.10 Solute Movement
A.11 Theis Equation
A.12 Glover Equation
A.13 Hantush-Jacob Equation
A.14 Weber Technique

A.1 Lowham Method

A.1.1 Summary

The Lowham method (Lowham 1976) is a basin characteristics method of estimating mean annual and peak flood flow magnitudes for essentially natural drainage basins in Wyoming encompassing 0.5 to 1,000 sq mi in area/extent. Basin characteristics methods are based on physiographic and climatic features of the basin.

A.1.2 Equations

The Lowham method uses regression equations to estimate mean annual flow, Qa, and peak flow, Qp, for various return periods. These equations are shown in Table A.1-1. The designations P2, P5, etc., refer to peak flow magnitudes having recurrence intervals of 2 years, 5 years, etc.

Elevation, E, is in thousands of feet and basin area, A, is in square miles.

A.1.3 Input Data

Required input data to utilize the Lowham Method are listed as follows:

- Hydrologic region or regions of Wyoming obtained from Figure A.1-1;
- Total drainage area;
- The percentage of basin area in each hydrologic region if the basin encompasses more than one hydrologic region; and
- Mean basin elevation if the basin is within Hydrologic Region 1 of Wyoming as indicated in Figure A.1-1.

A.1.4 Output Data

Lowham method regression equations and graphical relationships estimate peak flood flow and mean annual flow.

A.1.5 Limitations

The Lowham method is not applicable to sites where flood flows are affected by improvements such as major dams, reservoirs, diversions, or development.

The Lowham method should only be used to estimate peak flood flow magnitudes for basins within the range used to develop the procedure.

The Lowham method outputs only estimated peak flood flow magnitudes and does not produce a flood hydrograph.

A.1.6 Justification

The Lowham method requires little, if any, primary data and is useful for reconnaissance level hydrologic investigations of essentially natural basins where peak flood flow magnitudes are of primary interest and for which more detailed data are not required.

A.1.7 References

Lowham, H.W., 1976, Techniques for Estimating Flow Characteristics of Wyoming Streams, U.S. Geological Survey.

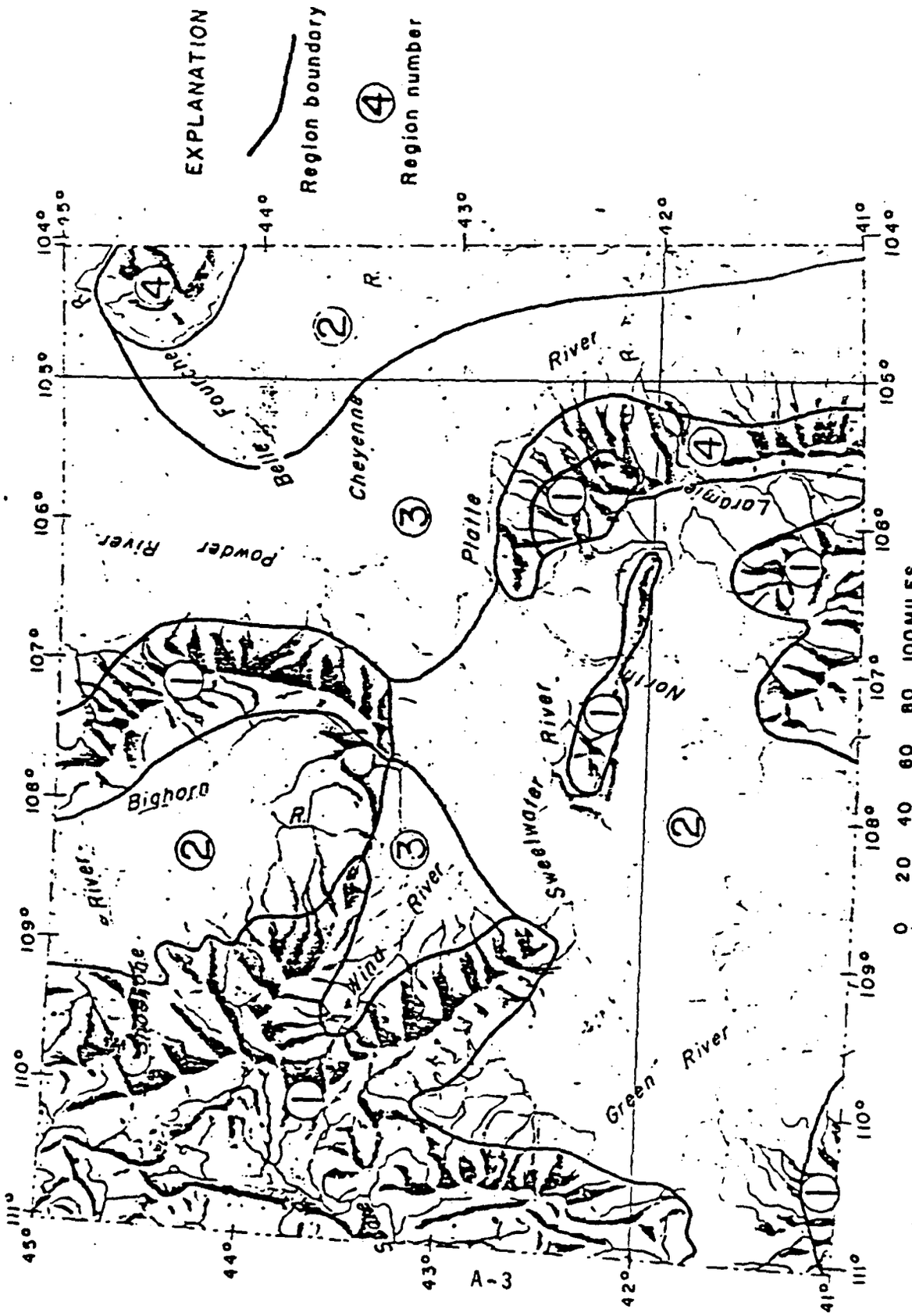


Figure A.1-1 Major streams and hydrologic regions of Wyoming.

Table A.1-1

MEAN ANNUAL FLOWS

Region	Regression equation (English units)
1	$Q_a = 0.0036\ A^{0.96}\ E^{2.57}$
2	$Q_a = 0.244\ A^{0.56}$ (intermittent and emphemeral streams)
3	$Qa = 0.518\ A^{0.53}$ (intermittent and ephemeral streams)
4	$Qa = 0.162\ A^{0.98}$ (perennial streams)

Note: Insufficient data concerning mean annual flow existed for intermittent and ephemeral streams in Regions 1 and 4, and for perennial streams in Regions 2 and 3; hence, no relations could be developed for these cases.

PEAK FLOWS

Region	Regression Equation (English units)			
1	P_2	=	0.009	$A^{0.85}E^{3.44}$
	P_5	=	0.070	$A^{0.81}E^{2.75}$
	P_{10}	=	0.203	$A^{0.79}E^{2.39}$
	P_{25}	=	0.634	$A^{0.77}E^{2.00}$
	P_{50}	=	1.32	$A^{0.76}E^{1.76}$
	P_{100}	=	2.57	$A^{0.75}E^{1.53}$
2[a]	P_2	=	56.8	$A^{0.38}$
	P_5	=	146	$A^{0.35}$
	P_{10}	=	239	$A^{0.34}$
	P_{25}	=	406	$A^{0.33}$
	P_{50}	=	572	$A^{0.32}$
	P_{100}	=	779	$A^{0.31}$

Table A.1-1 Continued, page 2 of 2

	PEAK FLOWS			
3[a]	P_2	=	93.8	$A^{0.36}$
	P_5	=	252	$A^{0.36}$
	P_{10}	=	425	$A^{0.37}$
	P_{25}	=	742	$A^{0.37}$
	P_{50}	=	1,070	$A^{0.37}$
	P_{100}	=	1,480	$A^{0.37}$
4	P_2	=	12.0	$A^{0.65}$
	P_5	=	36.3	$A^{0.61}$
	P_{10}	=	63.3	$A^{0.60}$
	P_{25}	=	115	$A^{0.58}$
	P_{50}	=	170	$A^{0.58}$
	P_{100}	=	242	$A^{0.57}$

Note: a Regression equations given for Regions 2 and 3 apply from 5 to 5,300 sq mi (13 to 13,700 sq km). Graphical review showed curvilinear relations exist from 0.5 to 5 sq mi (1.3 to 12 sq km). See graphs presented in Reference.

A.2 Technical Release 55 Method

A.2.1 Summary

Technical Release 55 approximates peak discharge results obtained from detailed computer modeling by means of simple graphical chart and tabular methods. These graphical chart and tabular methods were developed based on output data obtained from the Technical Release 20 computer program which is discussed in Section A.3 of this Appendix.

A.2.2 Equations

The graphical procedure of estimation peak flows presented in Technical Release 55 is to enter Figure A.2-1 with time of concentration and obtain peak discharge in cubic feet per second per square mile of drainage area per inch of runoff (csm/inch) and then multiply the csm/inch value by drainage area, A, in square miles and runoff, Q, in inches to obtain peak discharge.

To estimate time of concentration, T_c=5/3 L, and

$$L = \ell^{0.8} (S + 1)^{0.7} / 1900\, y^{0.5}$$

Where T_c = time of concentration, hours
L = lag time, hours
ℓ = hydraulic length of basin, feet
S = $(1{,}000/CN^1) - 10$
CN^1 = retardance factor which is equivilent to runoff curve number
y = average basin slope, percent

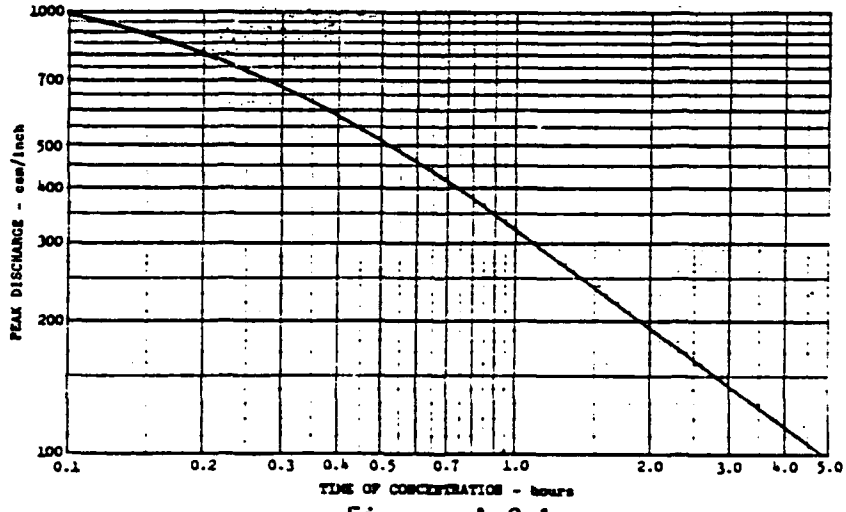


Figure A.2-1

Peak discharge in csm per inch of runoff versus time of concentration (T_c) for 24-hour, type II storm distribution.

To estimate runoff, Q:

$$Q = \frac{(P-0.2S)^2}{P+0.8S} \qquad \text{for} \quad P > 0.2S$$

$$Q = 0 \qquad \text{for} \quad P \leq 0.2S$$

$$S = \frac{1000}{CN} - 10 = I_a/0.2$$

Where:

Q = runoff, inches
P = precipitation, inches
S = maximum retention, inches
CN = curve number obtained from Table A.2-1
I_a = initial abstraction, inches

The chart procedure presented in Technical Release 55 is to obtain peak flow, in cubic feet per second, per inch of runoff from charts based on drainage area or equivalent drainage area, slope and runoff curve number. The peak flow obtained by the above procedure can be adjusted for various factors by use of charts and graphs in the references.

A.2.3 Input Data

Input data required to predict peak flows for both existing and future developed conditions using the methods and procedures presented in Technical Release 55 are listed as follows:

- Drainage area(s);
- Hydrologic soil type(s), percent of impervious area and corresponding runoff curve numbers for both existing and future developed conditions;
- Flow path data including length, slope, and hydraulic characteristics for both existing and future developed conditions; and
- Rainfall data for each storm recurrence interval under investigation.

Table A.2-1 Runoff Curve Numbers for Hydrologic Soil-Cover Complexes

(Antecedent Moisture Condition II, and $I_a = 0.2\ S$)

Land Use Description/Treatment/Hydrologic Condition			Hydrologic Soil Group			
			A	B	C	D
Residential:[1]						
Average lot size	Average % Impervious[2]					
1/8 acre or less	65		77	85	90	92
1/4 acre	38		61	75	83	87
1/3 acre	30		57	72	81	86
1/2 acre	25		54	70	80	85
1 acre	20		51	68	79	84
Paved parking lots, roofs, driveways, etc.[3]			98	98	98	98
Streets and roads:						
paved with curbs and storm sewers[3]			98	98	98	98
gravel			76	85	89	91
dirt			72	82	87	89
Commercial and business areas (85% impervious)			89	92	94	95
Industrial districts (72% impervious)			81	88	91	93
Open Spaces, lawns, parks, golf courses, cemeteries, etc.						
good condition: grass cover on 75% or more of the area			39	61	74	80
fair condition: grass cover on 50% to 75% of the area			49	69	79	84
Fallow	Straight row	---	77	86	91	94
Row crops	Straight row	Poor	72	81	88	91
	Straight row	Good	67	78	85	89
	Contoured	Poor	70	79	84	88
	Contoured	Good	65	75	82	86
	Contoured & terraced	Poor	66	74	80	82
	Contoured & terraced	Good	62	71	78	81
Small grain	Straight row	Poor	65	76	84	88
		Good	63	75	83	87
	Contoured	Poor	63	74	82	85
		Good	61	73	81	84
	Contoured & terraced	Poor	61	72	79	82
		Good	59	70	78	81
Close-seeded legumes[4] or rotation meadow	Straight row	Poor	66	77	85	89
	Straight row	Good	58	72	81	85
	Contoured	Poor	64	75	83	85
	Contoured	Good	55	69	78	83
	Contoured & terraced	Poor	63	73	80	83
	Contoured & terraced	Good	51	67	76	80
Pasture or range		Poor	68	79	86	89
		Fair	49	69	79	84
		Good	39	61	74	80
	Contoured	Poor	47	67	81	88
	Contoured	Fair	25	59	75	83
	Contoured	Good	6	35	70	79
Meadow		Good	30	58	71	78
Woods or Forest land		Poor	45	66	77	83
		Fair	36	60	73	79
		Good	25	55	70	77
Farmsteads		---	59	74	82	86

[1] Curve numbers are computed assuming the runoff from the house and driveway is directed towards the street with a minimum of roof water directed to lawns where additional infiltration could occur.

[2] The remaining pervious areas (lawn) are considered to be in good pasture condition for these curve numbers.

[3] In some warmer climates of the country a curve number of 95 may be used.

[4] Close-drilled or broadcast.

A.2.4 Output Data

Methods and procedures presented in Technical Release 55 output the following data for both existing and future developed conditions.

- o Runoff volumes;
- o Hydrographs; and
- o Modification of peak discharge due to development.

A.2.5 Limitations

Methods and procedures presented in Technical Release 55 do not adequately account for stormwater detention within a basin. The necessary computations become unwieldy for complex basins having many sub-basins or areas with different runoff characteristics.

The graphical procedure for estimating peak flood flows presented in Technical Release 55 satisfactorily approximates the results of detailed unit hydrograph calculations made either by hand or the Technical Release 20 computer program provided:

- o The initial rainfall abstraction is less than 25 percent of the total 24-hour rainfall;
- o Drainage subareas do not encompass more than 20 sq mi; and
- o There are not large changes in soil type and land use characteristics within the basin.

The chart procedure presented in Technical Release 55 is limited to drainage areas encompassing less than 2,000 acres.

To use the tabular method, individual sub-basins within the watershed should be less than 20 sq mi, there should be little variation in runoff curve number within each sub-basin, and rainfall should be sufficient to result in at least 1.5 inches of runoff.

A.2.6 Justification

The graphical procedure presented in Technical Release 55 provides a simple method of estimating peak flow where routing is not required and the land use, soil types, and cover are uniformly distributed throughout the basin.

The chart procedure presented in Technical Release 55 provides a simple method for assessing the effect of changing land use within small (less than 2,000 acre) basins.

The tabular procedure presented in Technical Release 55 provides a means of developing composite hydrograph data and assessing the effect of changing land use within one or more sub-basins on peak flows.

A.2.7 References

"Urban Hydrology for Small Watersheds"
Technical Release No. 55
U.S. Department of Agriculture
Soil Conservation Service
January 1975

National Engineering Handbook
Section 4, Hydrology
U.S. Department of Agriculture
Soil Conservation Service
August 1972

"A Guide to Hydrologcic Analysis using SCS Methods"
Richard H. McCuen
Prentice Hall 1982

A.3 Technical Release 20

A.3.1. Summary

Technical Release 20 (TR-20) computer program computes surface runoff from rainfall data and combines flood flow hydrographs from various tributaries and then routes flood hydrographs through stream channels and reservoirs.

A.3.2. Equations

TR-20 is based on Unit Hydrograph Theory (Sherman 1932) in conjunction with flow routing.

A brief summary of unit hydrograph procedures are listed as follows:

- o Determine the amount of rainfall during each interval;
- o Determine the amount of runoff (rainfall minus abstractions) during each time interval;
- o Develop a unit hydrograph, corresponding to 1 inch of runoff, which reflects all factors influencing runoff and flow within the basin;
- o Multiply the unit hydrograph by the amount of runoff during each time interval;
- o Develop basin or sub-basin hydrologic design point storm hydrograph by super position of individual runoff hydrographs; and
- o Combine sub-basin hydrographs by routing, if necessary.

U.S. Soil Conservation Service procedures to estimate runoff from rainfall based on runoff curve number are presented in Section A.2.2 of this appendix.

U.S. Soil Conservation Service equations to develop a triangular unit hydrograph corresponding to 1 inch of runoff are presented as follows:

$$L = \ell^{0.8} (S + 1)^{0.7} / 1{,}900 y^{0.5}; \quad \tau_c = 5/3\ L$$

$$D = 0.133\ \tau_c$$

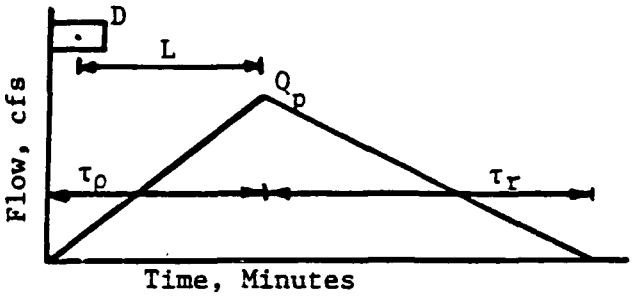
$$\tau_\rho = D/2 + 0.6\ \tau_c$$

$$\tau_r = 5/3\ \tau_\rho$$

$$Q_p = 484\ A\ Q/\tau_\rho$$

where,

- L = lag time, hours
- A = basin area, square miles
- ℓ = hydraulic length of area, feet
- y = slope of area, percent
- τ_c = time of concentration, hours
- D = duration of rainfall excess, or length of interval of rainfall excess, in hours
- τ_ρ = time to peak runoff, hours
- τ_r = recession time of hydrograph, hours
- Q_p = peak flow rate, cfs, with relationships shown below



The stream flow routing equation used by Technical Release 20 is:

$$Q_{\tau+\Delta\tau} = (1 - C)\, Q + CI,\ C = \Delta\tau/K$$

where,

$Q_{\tau+\Delta\tau}$ = flow at downstream end of channel reach at time $\tau + \Delta\tau$

Q = flow at downstream end at time τ

I = inflow at upstream point at time τ

C = routing coefficient

K = reach travel time estimated by dividing reach length by channel velocity, V, as estimated by the Manning equation.

Note: TR-20 uses $C = V/(V + 1.7)$ as default value

For reservoir routing, Technical Release 20 uses the storage equation in conjunction with site-specific stage storage and inflow data to determine the outflow hydrograph.

$$I - O = \Delta S/\Delta\tau$$

where,

I = inflow
O = outflow
$\Delta S/\Delta\tau$ = change in storage with respect to time

The storage equation is also used where a control structure restricts flow (O = fixed capacity).

A.3.3 Input Data

Input data required to use the TR-20 computer program include the following:

- Drainage basin area(s);
- Hydrologic soil types and corresponding soil cover complex number;
- Flow path data, including length, slope roughness, and cross section;

- o Stage-storage relationships for hydraulic structures;
- o Rainfall data for each storm recurrence interval under consideration;
- o Channel lengths, cross sections, and end area; and
- o Unit hydrograph data.

A.3.4 Output Data

The TR-20 computer program outputs hydrograph data, peak discharge, time of occurrence, and water surface at each cross section or structure desired.

A.3.5 Limitations

General limitations of the TR-20 program are listed as follows:

- o Does not compute backwater profiles;
- o Requires primary data for stage discharge and stage-storage curves;
- o Does not model snowmelt runoff; and
- o Does not model storm sewer networks.

A.3.6 Justification

The TR-20 computer program accommodates complex basins that would be unwieldy to analyze manually. Extensive storm drainage improvements warrant primary data collection in conjunction with a computer program, such as TR-20, to compute design flood flows because of the magnitude of investment required to construct and maintain extensive storm drainage improvements.

A.3.7 References

U.S. Department of Agriculture, Soil Conservation Service, "Computer Program for Project Formulation, Technical Release 20."

McCuen, R. H., "A Guide to Hydrologic Analysis Using SCS Methods", Prentice-Hall, Inc., Englewood Cliffs, New Jersey.

Sherman, L. K., "The Hydraulics of Surface Runoff," Civil Engineering, 10:165-166, 1940.

A.4 HEC-1 Flood Hydrograph Package

A.4.1 Summary

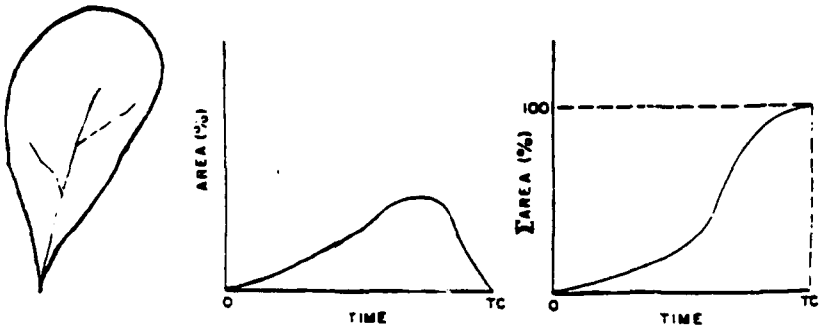
The HEC-1 program is used to perform flood hydrograph computations. It consists of several program packages, which are operated either individually or in combination, for the following flood hydrograph analysis:

- o Optimization of routing parameters;
- o Optimization of unit hydrograph and loss rate parameters;
- o Generalized precipitation, runoff, routing, and combining operations to simulate the hydrologic response of a watershed and its stream network;
- o Multiple flood computation; and
- o Multiple basin plan evaluation.

A.4.2 Equations

The HEC-1 program uses the Clark method for generating unit hydrographs when the unit hydrographs are not directly inputed into the model. In the application of Clark's unit hydrograph method, it is necessary to utilize a time-area curve for the basin under study. This curve may be obtained using measured or calculated flow travel times from various locations in the basin.

The figure below shows a typical basin, its time-area curve, and its time-accumulated area curve.



It is convenient to use a function of the form:

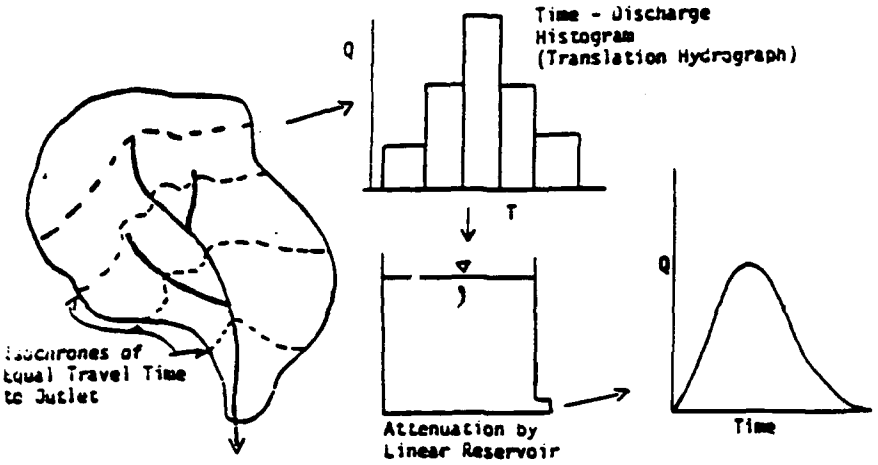
$$\text{Acc Area} = cT^n, \quad (o < T < Tc/2)$$

$$1- \quad \text{Acc Area} = c\,(1-T)^n, \quad (Tc/2 < T < Tc)$$

where

Acc Area = Accumulated Area
T = time
c,n = constants

The figure below shows the conceptional model of the Clark method:



The following equations are used to derive the Clark's unit hydrograph:

(a) Converting the time-accumulated area curve to time discharge curve

$$I_i = Ka_i / \Delta t$$

where

I_i = ordinate in unit volume of discharge (cfs) of the time discharge curve

K = conversion factor to convert inch-sq mi/hour to cfs (K=645)

a_i = ordinate in units of depth-area (inches-sq mi) excess of the time-accumulation area curve at the end of period i

Δt = computation interval in hours

(b) Routing the time-discharge curve through a linear reservoir

$$O_i = cI_i + (1-c)\, O_{i-1}$$

$$c = \frac{2\Delta t}{2R + \Delta t}$$

where

O_i = outflow from the basin at the end of the period

c = dimensionless routing constant

R = attentuation constant, having the dimension of time. The magnitude of R can be approximately evaluated at the point of inflection of the recession link of the observed surface runoff hydrograph by the expression $R = -\frac{Q}{dQ/dt}$

O_{i-1} = outflow from the basin at the end of period i-1

(c) Clark's unit hydrograph

$$Q_i = \frac{1}{n}(0.5\ O_{i-n} + O_{i-n+1} + \cdots + O_{i-1} + 0.5\ O_i)$$

where

O_i = ordinate at time i of unit hydrograph of duration D, and time interval t

$$n = \frac{D}{\Delta t}$$

The hydrologic methods of flood routing used by the HEC-1 program do not attempt a direct, complete solution of the differential equations that describe unsteady flow. These methods solve the continuity equation and a much simplified version of the energy equation. The methods generally employ semiempirical coefficients that must be computed by calibration. The hydrologic routing methods used by HEC-1 include the Modified Pulse, working R&D, and Muskingum methods. Also included in this group are the simpler averaging and lagging methods, such as the Successive Average-Lag (Tatum) and Progressive Average-Lag (straddle-stagger) methods. Each one of these methods is represented by a variation of the following general functional form:

$O(t+1) = f[Q(t), I(t+1), I(t), S]$

$O(t+1)$ = Outflow at time t+1
$O(t)$ = Outflow at time t
$I(t+1)$ = Inflow at time t+1
$I(t)$ = Inflow at time t
S = Storage/outflow table
f = Function of

A.4.3 Input Data

Depending upon which HEC-1 package options are used, the following input data are required:

- Observed hydrograph to be reconstructed;
- Routing optimization criteria and observed tributary inflow hydrographs;
- Unit hydrograph and loss rate optimization criteria;
- Station precipitation data;
- Multiflood data (flood and precipitation ratios);
- Hydrograph balancing criteria;

- o Subarea runoff computation data including precipitation, losses, and unit hydrograph information; and

- o Individual stream reach routing criteria.

A.4.4 Output Data

- o For optimization of the unit hydrograph, the best fit values of the hydrograph variables are printed along with an interval by internal listing of the precipitation, losses (inflow and storage for routing) and computed and observed discharges.

- o For watershed response simulations, the output provides precipitation, rainfall and snowmelt losses, and excess unit hydrographs, subarea outflow hydrographs, routed hydrographs, and combined hydrographs.

- o Graphical display of intermediate or summary hydrographs and precipitation can be called where desired.

- o For multiflood simulations, an interpolated hydrograph is printed for each stream system based on the size of the area tributary to that point.

- o For multiple basin plan evaluation runs, average annual damages are provided for each damage center for each plan of development as well as for existing conditions.

A.4.5 Limitations

The flood hydrograph package is applicable only to single-storm analyses. There is no provision for recovery of loss rate potential during periods of no precipitation. Numerous empirical coefficients are used in the program calibration, verification of which will require a large primary data base and efforts. HEC-1 does not adequately model runoff from impervious areas.

A.4.6 Justification

HEC-1 is more comprehensive than Technical Release 20 and includes the following features:

- o Analysis of actual precipitation data;

- o Snowmelt;

- o Optimization of hydrologic parameters; and

- o Economic analysis.

In addition, HEC-1 is commonly accepted for modeling the hydrology of large watersheds.

A.4.7 References

- U.S. Army Corps of Engineers, 1973, HEC-1 Flood Hydrograph Package Users Manual, The Hydrologic Engineering Center, Computer Program 723-X-6-L2010.

- U.S. Army Corps of Engineers, 1959, Engineering and Design, Flood Hydrograph Analyses and Computations, EM 1110-2-1411.

- U.S. Weather Bureau and U.S. Army Corps of Engineers, 1956, Seasonal Variation of Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Durations of 6, 12, 24, and 48 Hours, Hydrometerological Report No. 33, Washington, DC.

- U.S. Army Corps of Engineers, 1960, Engineering and Design, Routing of Floods Through River Channels, EM 1110-2-1408.

- Beard, Leo R., 1966, Optimization Techniques in Hydrologic Engineering, The Hydrologic Engineering Center, Technical Paper No. 2.

A.5 HEC-2 Water Surface Profiles

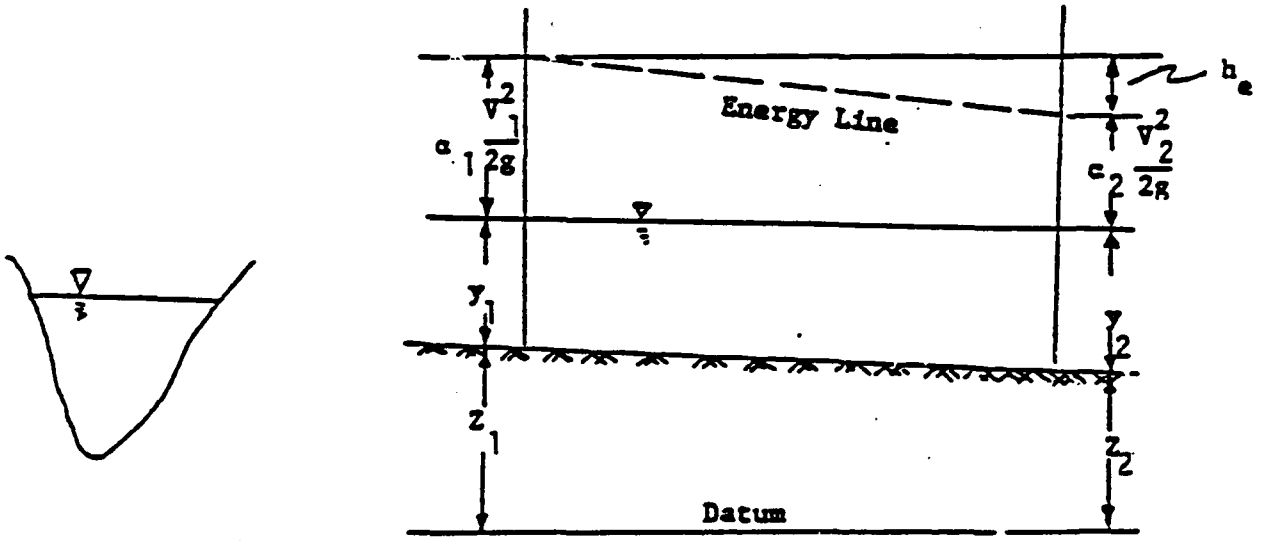
A.5.1 Summary

The HEC-2 water surface profile program computes steady flow water surface profiles for stream reaches. Special consideration is given to bridges, culverts, weirs, embankments, and dams. The program allows variable roughness, islands, bends, levee overflow, river confluences, and waterfalls. Channel roughness can be established from known highwater mark, if desired.

A.5.2 Equations

(a) One-dimensional energy equation

$$Z_1 + Y_1 + \alpha_1 \frac{V_1^2}{2g} = Z_2 + Y_2 + \alpha_2 \frac{V_2^2}{2g} + h_e$$



where,

h_e = energy loss between Sections 1 and 2
Z = ground elevation of the channel bottom
y = water surface elevation
v = velocity
g = gravitation constant
α = velocity distribution correction factor

(b) Steady flow equation

$$Q_1 = Q_2$$

where

$Q_{1,2}$ = flow discharge at Sections 1 and 2

(c) Energy losses

(i) Total loss

$$h_e = h_f + h_o$$

where,

h_f = energy loss due to friction

h_o = other losses

(ii) Friction loss in channel reach

$$hf = \bar{S}_f \cdot L; \quad \bar{S}_f = (S_{f_1} + S_{f_2}) / 2$$

$$S_f = (Q/K)^2; \quad h_f = L \left[\frac{2Q}{K_1 + K_2} \right]^2$$

where,

$\bar{S}_f$ = average frictional loss, frictional slope
Q = total water discharge
K = total conveyance factor
$= \frac{Cm}{n} \cdot AR^{2/3}$
Cm = constant, 1.486 for English unit
n = Manning's hydraulic roughness value
A = cross-sectional area
R = hydraulic radius
L = reach length

(iii) Contraction and expansion losses

$$h_o = C_c \left[\alpha 1 \frac{V_1^2}{2g} - \alpha 2 \frac{V_2^2}{2g} \right]$$

and,

$$h_o = C_e \left[\alpha 1 \frac{V_1^2}{2g} - \alpha 2 \frac{V_2^2}{2g} \right]$$

where,

C_c = empirical contraction coefficient
C_e = empirical expansion coefficient
$\alpha 1 \frac{V_1^2}{2g}$ = upstream velocity head

$\alpha 2 \frac{V_2^2}{2g}$ = downstream velocity head

A.5.3 Input Data

- o Flow, usually flood peak discharge;
- o Flow regime - either subcritical and supercritical flow;
- o Starting water surface elevation;
- o Energy loss coefficients;
- o Geometry of each cross section; and
- o Reach lengths.

A.5.4 Output Data

- o Depth of flow;
- o Water surface and mean energy gradient elevations;
- o Mean velocity head;
- o Energy loss data;
- o Total flow;
- o Amount of flow in left overbank, right overbank, and channel;
- o Cross-section area of left overbank, right overbank, and channel;
- o Cumulative volume of water in the river since the first cross section;
- o Cumulative top area of the river since the first cross section;
- o Travel time from first cross section to present cross section;
- o Mean velocity in right overbank, left overbank, and channel;
- o Manning's "n" for left overbank, right overbank, and channel area;
- o Weighted value of Manning's "n" for the channel based on the distance between cross section. Used when computing Manning's "n" from highwater marks;
- o Minimum elevation in the cross section;
- o Slope of energy grade line;
- o Distances in the left overbank, right overbank, and channel between the previous cross section and the current cross section;
- o Number of trials required to:

- Balance the assumed and computed water surface elevations;
- Determine critical depth; and
- Determine the water surface elevation by the slope area method or to balance the energy gradient in the special bridge routine.

o Area of the bridge deck subtracted from the total cross-sectional area in the normal bridge routine;

o Cross-section width at the assumed water surface elevation;

o Energy grade line elevations computed assuming either pressure flow or low flow control;

o Drop in water surface elevation from upstream to downstream sides of the bridge computed using Yarnell's equation assuming Class A low flow;

o Total weir and pressure flow at the bridge;

o Elevations of the bridge low chord and top of roadway;

o Net area of bridge opening below the low chord;

o Types of controlling flows;

o The stations where the water surface intersects the ground on the left and right side; and

o Left and right bank elevations.

A.5.5 Limitations

The area of application is limited by the following program assumptions:

o Channels of small slope (<1:10);

o Steady flow;

o One-dimensional flow; and

o Gradually varied flow (no acceleration, parallel streamlines).

A.5.6 Justification

It can be used to assess the changes in water surface elevation (floodplain delineation) and flow velocity as a result of the changes in tributary inflows. These output data provide quantitative measure of impacts due to a large increase in tributary inflows.

A.5.7 References

U.S. Army Corps of Engineers, 1973, "HEC-2 Water Surface Profiles Users Manual," The Hydrologic Engineering Center, Computer Program 723-X6-L202A.

U.S. Army Corps of Engineers, 1973, "HEC-2 Water Surface Profiles Programmers Manual," The Hydrologic Engineering Center, Computer Program 723-X6-L202A.

A.6 Urban Runoff Method

A.6.1 Summary

This method provides a rough estimate of stormwater quantity and quality for urban runoff that relies on national data to establish pollutant loads.

A.6.2 Equations

Quantity

1. Determine rainfall rate for 1 year frequency, 30 minute duration rainfall event. Determine runoff rate as follows:

   Runoff rate = K (rainfall rate) where K = 0.8 for slopes <2%, 0.85 for slopes 2-7%, and 0.9 for slopes >7%.

2. Develop synthetic hydrograph

$$T_R = 13.12\ L^{0.315}\ S^{-0.0488}\ I^{-0.490}$$

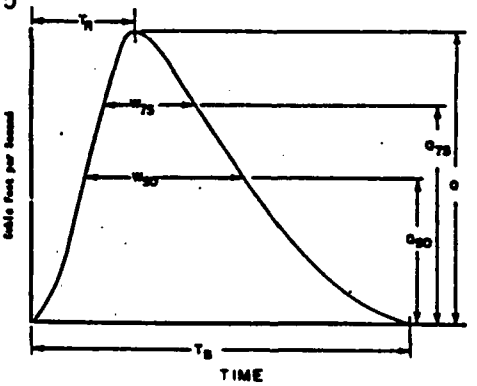
$$Q = 3.54 \times 10^4\ T_R^{-1.10}\ (A/640)$$

$$T_B = 3.67 \times 10^5\ (A/640)^{1.14} Q^{-1.15}$$

$$W_{50} = 4.14 \times 10^4\ (A/640)^{1.03}\ Q^{-1.04}$$

$$W_{75} = 1.34 \times 10^4\ (A/640)^{0.92}\ Q^{-0.94}$$

Where: A is the size of the study area or subarea in acres,
L is the length of the main drainage channel in feet,
S is the slope of the main drainage channel in feet/feet,
I is the percent of impervious cover for the study area or subarea, and
T_R, Q, T_B, W_{50}, W_{75} are shown below:



3. Separate hydrograph into impervious and pervious fractions.

   a. Estimate area in streets, slope, and drainage length.
   b. Estimate off-street impervious area, slope, and drainage length.
   c. Calculate hydrographs as in 2 above.

4. The unit hydrograph is based on 1 inch of runoff in 0.5 hour. For a different runoff rate multiply ordinate values of graph by design runoff rate per 2 inches per hour.

Quality

The analysis assumes loadings on impervious surface (primarily streets) is responsible for pollutants:

$$L_C = A_R \cdot D_A$$

Where L_C = solids loading (lb/curb mile)
A_R = rate of accumulation (lb/curb mile/day)
D_A = equivalent days of accumulation (days)

$$D_A = (D_R - D_S)(1 - E_S) + D_S$$

Where D_R = days since last substantial rainfall
D_S = days since last street sweeping
E_S = street sweeping efficiency (say 40%)

Removal of solids is calculated as follows:

$$P_o - P = P_o (1 - e^{-4.6rt}), \text{ where:}$$

P_o = amount of pollutant on surface at beginning of storm
$P_o - P$ = amount of pollutant washed away in time, t
P = amount of material remaining on surface
r = rate of runoff (inches/hr)

The factor of 4.6 was derived by assuming 90 percent of pollutants would be washed from the surface in 1 hour of runoff of 0.5 inch per hour.

The hydrograph for streets is divided into equal units of time and the volume of runoff for a time period is calculated as the area under the hydrograph for that unit of time. The amount of material washed away is calculated as above. The concentration in the runoff is the amount of material washed away divided by the volume of total runoff under the hydrograph. The process is repeated for the next step. Values for other parameters are based on national relationships between the micrograms of pollutant contained in a gram of dry solids.

A.6.3 Input Data

- Area of drainage;
- Land use in drainage;
- Length of drainage channels;

- o Slope of drainage channels;
- o Percent impervious surface; and
- o Design rainfall.

A.6.4 Output Data

Hydrograph of runoff and pollutograph of water quality constituents.

A.6.5 Limitations

Results of applying this technique have only been marginally successful in simulating measured storm runoff quantities and characteristics. This method relies on national data that may not be locally appropriate.

A.6.6 Justification

Little input data requried for model application. Results can be used to assess need for more sophisticated methods (SWMM).

A.6.7 References

Amy, et. al, 1974, "Water Quality Management Planning for Urban Runoff" EPA-440/9-75-004, for Environmental Protection Agency, Washington DC.

Santor and Boyd, 1972, "Water Pollution Aspects of Street Surface Contaminants" EPA-R2-12-081 for Environmental Protection Agency, Washington DC.

A.7 Storm Water Management Model (EPA-SWMM)

A.7.1 Summary

The U. S. Environmental Protection Agency's Stormwater Management Model (EPA-SWMM) computes the combined storm and sanitary runoff from multiple catchments and routes the flow through a converging branch of sewers network. Flow diversion structures can be modeled and storage can be simulated for both inline and overflow retention basins. Both dry-weather and stormwater quality for suspended and settleable solids, biochemical and chemical oxygen demand, coliform bacteria, phosphorus, nitrogen, and oil and grease are computed for each modeled catchment and routed through the sewerage system.

A.7.2 Equations

a. Runoff quantity calculation

i. Infiltration

$$I = k_1 + (k_2 - k_1)\, e^{-k_3 t}$$

Where I = infiltration to groundwater during t
k_1, k_2 = minimum and maximum infiltration rates, respectively
k_3 = exponential rate of loss in infiltration capacity

ii. Surface runoff

$$Q_0 = \frac{1.49}{n}\, S^{1/2}\, W \left(\frac{d_0 + d_1}{2} - d_s \right)^{5/3}$$

Where Q_0 = outflow from catchment
n = Manning coefficient
W = width of the catchment plane
S = slope of the catchment
d_0 = flow depth at time t
d_1 = flow depth at time, $t + \Delta t$
d_s = maximum depth of detention storage

iii. Continuity equation

$$\frac{\Delta d}{\Delta t} = \left[R - I + \frac{Q_i - Q_0}{As} \right]$$

Δd = an incremental change in flow depth per time step, t
R = rainfall during t
Q_i = inflow from upsteam catchments
As = surface area of element

b. Runoff quality calculation

i. Mass loading on the surface

$$P = f(K, D, G, N)$$

Where
- P = total mass on watershed at start of storm
- K = mass of constituent per unit mass of dust and dirt (a function of land use type)
- D = rate of dust and dirt accumulation (also a function of land use)
- G = total length of gutters
- N = number of days since last storm or street cleaning

ii. Mass washoff

$$\frac{dP}{dt} = KP$$

Where
- P = mass of pollutant on the catchment
- K = runoff coefficient
- $=$ br
- b = proportionality coefficient
- r = rate of surface runoff

c. Flow routing in the sewer network

i. Routing dictated by the flow-area relationship of the conduits.

The flow-area data are generated from Manning's equation, normalized by dividing by the corresponding equation for the conduit flow full, denoted by the subscript, f thus,

$$Q/Q_f = \frac{A(R)^{2/3}}{A_f(R_f)^{2/3}} = f(A/A_f) = g(y/y_f)$$

Where
- Q = flow, cfs
- A = flow area, ft^2
- R = hydraulic radius, ft
- y = flow depth, ft
- f, g = functions

ii. Routing based on the equation of motion and continuity the general motion equation.

$$\frac{\partial Q}{\partial \tau} = -gAS_f + 2u\frac{\partial A}{\partial \tau} + u^2\frac{\partial A}{\partial X} - gA\frac{\partial H}{\partial X}$$

Where
- Q = discharge
- u = velocity
- A = cross-sectional area of flow
- H = hydraulic head
- S_f = friction slope

x = distance along conduit
τ = time

The continuity equation for a junction node is

$$\frac{\partial H}{\partial t}_t = \frac{\Sigma Qt}{Ast}$$

Where ΣQt = sum of all inflows to this node
Ast = water surface area associated with the node

d. Transport equation for the water quality constituents

In terms of mass rate of change

$$\frac{\partial M}{\partial t} = QC$$

Where M = mass
C = concentration

In terms of rate of change in concentration

$$\frac{\partial C}{\partial t} = -\frac{C}{V}\frac{\partial V}{\partial t} + u\frac{\partial C}{\partial X}$$

V = volume

A.7.3 Input Data

a. For runoff calculation

i. Quantity

- Rainfall hyetograph pertinent to the watershed;
- A watershed identification number and the sewer node to which it drains;
- The size and length of each unit watershed (catchment); and
- The average ground slope, hydraulic resistence.

ii. Quality infiltration coefficients

- The number of dry days prior to the onset of rainfall. The street cleaning frequency, and the equivalent gutter length and number of catch basins for each unit watershed.
- Land use type (a single type for each catchment) five land uses which may be modeled are: single-family residential, multifamily residential, commercial, industrial, and undeveloped or parklands.

- o Dust and dirt buildup rate (pounds per day per 100 feet of curb) to each land use type.
- o Mass of constituent per unit mass of dust and dirt for each land use type.

b. For transport calculation

i. Conduits

- o Conduit number;
- o Junctions which the conduit connects;
- o Conduit type (circular, rectangular, etc.);
- o Cross-sectional area;
- o Conduit length;
- o Maximum width of conduit;
- o Conduit length;
- o Manning's "n" value;
- o Distance of conduit invert above nodal elevation if different from zero; and
- o Head loss coefficients at junctions.

ii. Junctions

- o Junction number;
- o Ground elevation at the node;
- o Invert elevation at the node; and
- o Net value of constant inflow to the node, if any.

A.7.4 Output Data

a. Runoff simulation

Output from single event runoff simulation consists basically of hydrographs and pollutographs printed over the whole event at a specific interval of time steps. Continuous SWMM retains this option for up to five user-specified date intervals. In addition, daily, monthly, annual and grand total value for runoff, precipitation, and pollutant loads are provided.

b. Transport simulation

Output from transport simulation provides routed hydrograph and pollutograph which are printed over the whole event at a specific interval of time steps. The output also can provide temporal and spatial profile of the flow depth in the transport network.

A.7.5 Limitations

- The model requires a large data base (e.g., flow, pollutant concentrations) and substantial time and effort for calibrating and verifying the many empirical coefficients.
- The model is applicable only in the urban area, especially the overland runoff portion of the program.
- The model is relatively more expansive to run than HEC-1 or Technical Report 20 models.
- The model only approximates the backwater effect in the sewer network. If surcharging occurs, SWMM assumes that all water in excess of full pipe flow capacity is stored in the next upstream manhole and the surcharged volume does not propagate farther upstream.
- The quality portion of the program, especially the loading and washoff computation routine's, at the best grossly approximate a runoff event. To improve the accuracy, these routines may need to be modified to represent the site-specific conditions.

A.7.6 Justification

- The model allows the quantity and quality analysis of an urban sewer network.
- It is a comprehensive model, calculating both dry and wet weather flows in the sewer system.
- It provides data for sewer design.
- It is widely accepted by the regulatory agencies.

A.7.7 References

- U.S. Environmental Protection Agency, 1982, Storm Water Management Model-User's Manual, Version III.
- Metcalf and Eddy, Inc., University of Florida and Water Resource Engineers, 1971, Storm Water Management Model, Volume III-User's Manual, U.S. Environmental Protection Agency, Water Publity Office, Report No. 11024 DOC 09/71.

A.8 Modified Streeter-Phelps Equation

A.8.1 Summary

Maintenance of adequate dissolved oxygen (DO) levels is one of the most important considerations in determining allowable wastewater discharges. Various models have been developed to describe DO relations in streams. A modified version of the Streeter-Phelps equation is described here.

A.8.2 Equations

The equations used for the prediction of stream dissolved oxygen (DO), carbonaceous biochemical oxygen demand (BOD), and NH -N are as follows:

$$D(t) = \frac{K_1 L_0}{K_2 - K_1}\left(e^{-K_1 t} - e^{-K_2 t}\right) + \frac{K_n N_0}{K_2 - K_n}\left(e^{-K_n t} - e^{-K_2 t}\right) + D_0 e^{-K_2 t}$$

$$L(t) = L_0 e^{-K_1 t}$$

$$N(t) = N_0 e^{-K_n t}$$

Where
- $D(t)$ = DO deficit at time t
- $L(t)$ = Ultimate carbonaceous BOD at time t
- $N(t)$ = Nitrogenous BOD at time t, where Nitrogeneous BOD = 4.5 x NH_3 -N
- D_o = Initial DO deficit
- L_o = Initial ultimate carbonaceous BOD
- N_o = Initial nitrogenous BOD
- K_1 = Carbonaceous deoxygenation rate constant
- K_n = Nitrogenous deoxygenation rate constant
- K_2 = Reaeration rate constant

The initial DO deficit D equals saturated value for oxygen minus initial concentration and its determination involves two calculations:

$$C_s = 24.89 - 0.426t + 0.00373t^2 - 0.0000133t^3$$

Where
- t = Water temperature, °F
- C_s = Saturation value for oxygen at t

DO concentration after mixing which is determined by the mass balance equation:

$$C = \frac{Q_w C_w + Q_s C_s}{Q_w + Q_s}$$

Where
- C = concentration after mixing
- Q_w = flow of discharge
- C_w = concentration of discharge
- Q_s = flow of stream upstream of discharge
- C_s = concentration of stream upstream of discharge

The same procedure of mass balance is used to determine L_o & N_o, other parameters are:

$$K_1 = 0.2 \times 1.047^{T-20}$$

$$K_n = 0.3 \times 0.058 \ (T-.16), \ T>3°C$$

$$K_2 = K_2 \ @20°C \times 1.0241^{T-20}$$

One of the methods to calculate K_2 is the O'Connor and Dobbins formula:

$$K_2 \ @20°C = \frac{12.9V^{1/2}}{d^{3/2}}$$

where V = mean stream velocity, fps
d = mean stream depth, ft

The Manning equation is used to calculate V

$$V = \frac{1.48}{n} R^{2/3} S^{1/2}$$

where
R = hydraulic radius, ft
S = channel slope
n = roughness coefficient

The following equation is used to calculate d

$$d = \left[\frac{VAn}{1.48 \ ws^{1/2}} \right]^{3/5}$$

where
A = cross-section area of stream, sq ft
w = water surface width, ft

A.8.3 Required Input Data

Data are required on flow and quality for both the stream and the wastewater discharge. The slope and channel characteristics of the stream are also required.

A.8.4 Output Data

The DO level at any point downstream.

A.8.5 Limitations

The method above ignores photosynthesis (algae) and benthic demands. The collection of accurate input data usually requires an extensive field program. The equations apply to steady-state, one-dimension streamflow with no instream dispersion.

A.8.6 Justification

This method would provide an initial assessment of the impact of waste discharges on the DO concentration in the receiving waters.

A.8.7 References

Metcalf and Eddy, 1972, Wastewater Engineering, McGraw-Hill Book Company, New York.

Tsivoglou and Wallace, 1972, "Characterization of Stream Reaeration Capacity," EPA-R3-72-012, U.S. Environmental Protection Agency.

A.9 U.S. Geological Survey Groundwater Model

A.9.1 Summary

The modified USGS two-dimentional groundwater flow model simulates the groundwater flow system of an area and can project the response of an aquifer to groundwater withdrawals and resulting changes in water levels and streamflow.

A.9.2 Equation

The two-dimensional groundwater flow equation is:

$$\frac{\partial}{x}\left(K_x b \frac{\partial h}{\partial x}\right) = \frac{\partial}{\partial y}\left(K_y b \frac{\partial h}{\partial y}\right) = S \frac{\partial h}{\partial t} + W$$

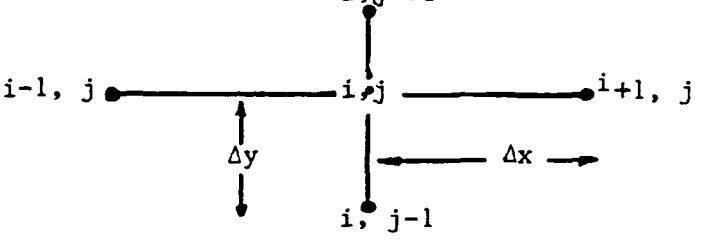
where:

b	=	saturated thickness
h	=	hydraulic head
K_x	=	hydraulic conductivity in the x-direction
S	=	storativity
t	=	time
W	=	source-sink term
x,y	=	spacial variables in the plane of the aquifer

The W factor includes all the flux inflows (precipitation, recharge) and outflows (pumping, discharge to streams) for an area. In application, the two-dimensional groundwater flow equation is approximated by discretization of the continuum. In the discretization, the region of flow is divided into a finite number of blocks, each with its own hydrogeologic properties, and each having a node at the center at which the hydraulic head is defined for the entire block.

$$\frac{\partial^2 h}{\partial h^2} = \frac{h_{i-1,j} + h_{i+1,j} - 2h_{i,j}}{\Delta x}$$

where:



The approximations are simulated in time using the strongly implicit procedure (Strong 1968).

A.9.3 Input Data

Input data required for application include:

- o Piezometric head at each node;
- o Hydraulic properties at each node (hydraulic conductivity, storativity, and initial saturated thickness);
- o Recharge, discharge, and "streamflow" at each node; and
- o Boundary conditions of the area being modeled.

A.9.4 Output data

The model calculates time-dependent changes in the piezometric head for each node in the area modeled.

A.9.5 Limitations

A digital model is an idealized mathematical approximation to a real hydrologic system, and because it is an approximation, it has limited capability to simulate in detail the behavior of the hydrologic system that it is intended to represent. The departure of the simulated behavior from the actual behavior is termed the error of the simulation. Error is introduced by the numerical procedures, specifically the finite-difference techniques, that are employed to solve the flow equation and by error associated with the input hydrologic field data.

Error is contributed by the field data through 1) limitations in the accuracy with which measurable quantities, such as the rate of discharge of water from the aquifer to streams, can be determined, and 2) through uncertainties associated with the indirect determination of properties, such as the hydraulic conductivity distribution.

A.9.6 Justification

Accuracy of the Crist (1980) Laramie County model to simulate the hydrologic system was determined by comparing the water-level changes measured at 37 observation wells in areas of irrigation pumping with the water-level changes calculated by the model for 1971 to 1977. On this basis, it is concluded that the model adequately simulates the hydrologic conditions in the post-Cretaceous rocks in Laramie County and may be used to predict the effect of applied stress to the system.

A.9.7 References

- o Crist, M. A., 1980, Effect of Pumpage on Groundwater Levels as Modeled in Laramie County, Wyoming, U.S. Geological Survey Water Resources Investigation, Open-File Report 80-1104.
- o Hoxie, D.T., 1977, Digital Model of the Arikaree Aquifer near Wheatland, Southeastern Wyoming, U.S. Geological Survey Open-File Report 77-676.

- Stone, H.K., 1968, Iterative solution of implicit approximations of multidimensional partial differential equations: Soc. Indust. Appl. Math., Jour. Numer. Anal., v. 5, no. 3, p. 530-558

- Trescott, P.C., Pinder, G.F., and Larson, S. P., 1976, Finite-difference numerical experiments: U.S. Geol. Survey Tech. Water-Resources Investigations, Bk. 7, Chap. C1, 116 p.

A.10 Solute Movement

A.10.1 Summary

Water or wastewater movement in both the unsaturated and saturated soils is quite complex, but can be approximated by use of hydrodynamic dispersion equations to account for advection (transport by bulk motion of the flowing groundwater) and diffusion (transport under the influence of concentration gradients).

A.10.2 Equations

The one-dimensional form of the advection-dispersion equation for nonreactive dissolved constituents in saturated, homogeneous, isotropic materials under steady-state uniform flow is:

$$D_x \frac{\partial^2 C}{\partial x^2} - \bar{v} \frac{\partial C}{\partial x} = \frac{\partial C}{\partial t}$$

Where x = is a curvilinear coordinate direction taken along the flowline

$\bar{v}$ = is the average linear groundwater velocity

C = is the solute concentration

D_x = is the coefficient of hydrodynamic dispersion along the flow path

and

$$D_x = \alpha \bar{v} + D^*$$

Where α = is the dispersivity.
D^* = is the coefficient of molecular diffusion.

As the contaminate mass is transported through the flow system, the concentration distribution of the contaminate mass at time t is given by:

$$C(z, t) = \frac{M}{8(\pi t)^{3/2} \sqrt{D_z}} \exp \frac{-z^2}{4D_z t}$$

Where M = is the mass of contaminate introduced
z = is the verticle distance from the center of gravity of the contaminate mass

Warrick et al. (1971) has shown that the water infiltration rate into a thick vadose (unsaturated) zone approaches a constant. The water front advance is given by:

$$\frac{\partial z}{\partial t}\theta = \frac{K(\theta_s)}{\theta s} \left[\frac{\theta_s}{\theta_s - \theta_j}\right]$$

Where θ_s = is the soil surface water content

z_θ = is the position of a given water content

θ_j = is the initial soil water content

$K(\theta_s)$ = is the unsaturated hydraulic conductivity

The water front advance would be at a faster rate than a nonreactive wastewater constituent. The latter movement is given by:

$$\frac{\partial z_\theta}{\partial t} = \frac{K(\theta_s)}{\theta_s}$$

Where z_θ is the position of a given concentration of the constituent.

A.10.3 Input Requirements

Input data required for application include;

- o Characteristics of chemical constituents;
- o Application rate and mass;
- o Soil surface water content;
- o Initial soil water content;
- o Thickness of the unsaturated zone;
- o Coefficient of diffusion;
- o Dispersivity; and
- o Unsaturated hydraulic conductivity.

A.10.4 Output

The techniques evaluate water content and concentrations of nonreactive constituents along the flow path (assumed vertical).

A.10.5 Limitations

The one-dimensional expression of transport of dissolved constituents would not accurately represent field conditions because dispersion occurs in the transverse directions as well as the longitudinal direction of flow. A three-dimensional analysis would require computer modeling with significant additional new input data. Many of the input parameters, even in the simple one-dimensional case, will need to be estimated. The chemical constituents must be dissolved and nonreactive.

A.10.6 Justification

The equations can be used to estimate the quantitative results of hypothetical field conditions.

A.10.7 References

Day, P. R., 1956, "Dispersion of a Moving Salt-Water Boundary," Transactions of American Geophysical Union, Vol. 39:7-10.

- Elzeftowy, A., 1982, Movement or Water and Nitrate-Nitrogen Through Unsaturated Sediments Below the Root Zone, Desert Research Institute.

- Freeze, R. A. and Cherry, J. A., 1979, Groundwater, Prentice-Hall Inc., New Jersey.

- Warrick, A. W., J. W. Biggar, and D. R. Nielson, 1971, Simultaneous Solute and Water Transfer for an Unsaturated Soil, Water Resources Research, Vol. 7:1216-1225.

A.11 Theis Equation

A.11.1 Summary

The Theis equation could be used to interpret or predict the hydrologic effects of a transient or changing aquifer system caused by well production. When a well penetrating an extensive confined aquifer is pumped at a constant rate, the influence of the pumping extends outward with time. The water comes from a reduction in storage within the aquifer. The head in the aquifer continues to decline as long as the aquifer is effectively infinite in areal extent. The rate of decline continually decreases as the area of influence expands.

A.11.2 Equation

Theis (1935) derived the equation for nonsteady, radial flow in an infinite confined aquifer from an analogy between heat flow and well hydraulics and incorporated a time factor and storativity of the aquifer. The Theis equation is:

$$s = \frac{Q}{4 \pi T} \int_{u}^{\infty} \frac{e^{-u}\, du}{u}$$

where $u = \frac{r^2 S}{4Tt}$

- s = drawdown in an observation well at a distance r from the pumped well
- Q = constant well discharge
- T = transmissivity of the aquifer
- S = storativity of the aquifer
- t = time since pumping started

The integral is commonly written as the function W (u) and is called the "well function of u." The well function can be evaluated by the series:

$$W(u) = 0.5772 - \text{Ln}(u) + u - \frac{u^2}{2(2!)} + \frac{u^3}{3(3!)} - \frac{u^4}{4(4!)} + \cdots$$

Tables for the values of W (u) versus u are available. In addition, graphs of W (u) versus u can be used to interpret field test data for evaluation of aquifer properties.

A.11.3 Input Data

To interpret aquifer parameters it is necessary to input pump test data. These data include observation well drawdown at various time intervals and the constant discharge rate of the pump well. To evaluate potential piezometric decline it is necessary to input the aquifer parameters, transmissivity, and storativity.

A.11.4 Output Data

Aquifer parameters or prediction of aquifer response to pumping a well.

A.11.5 Limitations

The assumptions that underlie the Theis solution are as follows:

- The aquifer is homogeneous and isotropic, has uniform thickness, and infinite areal extent;
- The aquifer is confined and has no recharge;
- The pumping well penetrates the full thickness of the aquifer, its diameter is very small, and it is pumped at a constant rate;
- Prior to pumping, the piezometric surface is horizontal; and
- The water removed from storage is discharged instantaneously with decline of head.

However, the hydrogeologic environment is seldom as ideal as outlined by the above assumptions.

A.11.6 Justification

The Theis equation can be used to evaluate aquifer parameters or conversely to calculate the hydraulic response to pumping a well. If the limitations are not fully met, then the Theis equation can be used to bound a solution.

A.11.7 References

- Bureau of Reclamation, 1977, Groundwater Manual, U. S. Department of the Interior.
- Theis, C. V., 1953, "Relation Between the Lowering of the Piezometric Surface and the Rate and Duration of Discharge of a Well using Ground-Water Storage," American Geophys. Union Trans., Part 2, pp 519-524, dupl. as U. S. Geol. Survey Ground Water Note 5, 1952.

A.12 Glover Stream Depletion Equation

A.12.1 Summary

The Glover stream depletion equation (1960) is a modification of the Theis equation to estimate surface water depletion resulting from operation of a nearby well.

A.12.2 Equation

The Glover stream depletion equation is defined as follows:

$$q' = Q \left[1 - \frac{2}{\sqrt{\pi}} \int_0^{x/\sqrt{4\alpha t}} e^{-u^2} du \right]$$

where:
- q' = stream depletion rate
- Q = well discharge rate
- x = distance from pumping well to river
  = $T/\emptyset$ an aquifer constant
- $\emptyset$ = aquifer porosity
- T = aquifer transmissivity
- t = time
- u = $r/\sqrt{4\alpha t}$
- r = distance to an observation well

The Glover stream depletion equation is best used to estimate the percentage of water pumped from a well that originates from a nearby stream for a specific duration of well discharge. The ultimate steady state is reached when $(x / 4 t)$ becomes zero. Under these conditions, the flow coming from the reach -z to +z is given by:

$$\int_{-z}^{z} fd = \frac{Q}{\pi x} \int_{-z}^{z} \frac{x \cdot d}{(x^2 + z^2)} = \frac{2Q}{\pi} \arctan \frac{z}{x}$$

where: d = length of reach

An infinite reach of stream would eventually supply nearly all of the water pumped from the nearby well.

A.12.3 Input Data

The Glover stream depletion equation requires the previous knowledge of aquifer parameters (transmissivity and porosity).

A.12.4 Output Data

The Glover stream depletion equation is best used to estimate the percentage of water pumped from a well that originates from a nearby stream for a specific duration of well discharge. This result may then be used to approximate surface water depletion.

A.12.5 Limitations

The transient equation is based on the following assumptions:

- o The aquifer is homogeneous, isotropic, of uniform thickness, and of infinite areal extent.
- o The discharging well completely penetrates and is open to the aquifer.
- o Flow to the well is radial and horizontal.
- o The stream can be idealized as an infinite straight line.
- o The stream is fully penetrating and of constant head.

The condition of a constant head, fully penetrating, infinite length, straight-line stream is likely never met. Recharge to the pumping well by any other source (i.e., precipitation) is not considered. Thus, the degree of applicability of the resulting stream depletion rate must be site specific.

A.12.6 Justification

The Glover stream depletion equation is a valid tool for use in a sensitivity analysis of well sitting near a stream. However, since field conditions rarely conform to the assumptions of this technique, the results must be used with caution.

A.12.7 Reference

- o Bureau of Reclamation, 1977, Groundwater Manual, U.S. Department of the Interior.
- o Glover, R.E., 1960. "Ground Water Surface Water Relationships" Water Research Conference, Colorado State University, Fort Collins.

A.13 Hantush-Jacob Equation

A.13.1 Summary

Hantush and Jacob (1955) derived the equation for nonsteady radial flow in an infinite leaky confined aquifer to overcome the inherent assumptions in the Theis solution that the confining beds overlying and underlying the aquifer are impermeable, that they release no water from storage, and that the vertical component of flow is negligible which are seldom satisfied. An aquifer can receive significant flow from, or lose water into an aquitard, and is referred to as a leaky aquifer.

A.13.2 Equation

The Hantush-Jacob equation can be presented in the same form as the Theis equation but with a more complicated well function:

$$s = \frac{Q}{4\pi T} W(u, r/B)$$

where:

$$u = \frac{r^2 S}{4Tt}$$

$$r/B = r\sqrt{K^1/Tb^1}$$

s = drawndown of an observation well at a distance r from the pumped well
Q = *constant well discharge*
T = transmissivity of the aquifer
S = storativity of the aquifer
t_1 = time since pumping started
K^1 = vertical hydraulic conductivity of the aquitard
b^1 = thickness of the aquitard

W(u,r/B) is known as the leaky well function.

A family of type curves of W(u,r/B) versus u at various r/B values are typically used to interpret field test data for evaluation of aquifer properties.

A.13.3 Input Data

To interpret aquifer parameters it is necessary to input pump test data. These data include observation well drawdown at various time intervals and the constant discharge rate of the pump well. To evaluate potential piezometric decline it is necessary to input the aquifer parameters; transmissivity, storativity, and leakage.

A.13.4 Output Data

Aquifer parameters or prediction of aquifer response to pumping a well.

A.13.5 Limitations

Many of the assumptions in the Hantush-Jacob method are the same as in the Theis method:

- The aquifer is homogeneous, isotropic, infinite in areal extent, and is of uniform thickness throughout.
- The well completely penetrates the aquifer, and flow in the aquifer is radial throughout.
- The aquifer is overlain by an aquitard of low but finite permeability which is in turn overlain by an unconfined aquifer. The aquifer is underlain by an aquaclude. Flow in the aquifer is augmented by vertical leakage through the aquitard.
- No water is released from storage in the aquitard.
- The overlying water table is not influenced appreciably by pumping.

A.13.6 Justification

The Hantush-Jacob solution is used in place of the Theis method in cases where recharge or vertical leakage cannot be ignored. Because of the presence of recharge, water levels will stabilize when the entire discharge of the well is derived from leakage.

A.13.7 References

- Hantush, M. S., and Jacob, C. E., 1955, "Nonsteady Radial Flow in an Infinite Leaky Aquifer" Am. Geophys. Union Trans., Vol. 36, No. 1, pp 95-100.
- Lohman, S. W., Ground-Water Hydraulics, U. S. Geological Survey professional paper 708.

A.14 Weber Technique

A.14.1 Summary

The Weber equilibrium equation is a modification of the Theim-Forchheimer steady-state equation to allow for uniform recharge to an unconfined aquifer. The equation is used to evaluate the steady state distance-drawdown relationship from a discharging well in an unconfined aquifer where the total flow to the well equals the recharge within the radius of influence.

A.14.2 Equation

The Weber equation is defined as follows:

$$s^1 - \frac{(s^1)^2}{2m} = \frac{Q}{2\pi T} \left[\ln (Q/\pi r^2 w) + \pi r^2 w/Q - 1 \right]$$

where:
- s^1 = drawdown observed in unconfined aquifer
- m = initial saturated aquifer thickness
- Q = well discharge
- T = aquifer transmissivity
- w = rainfall accretion

The Weber equation can be carried one step further to yield the radius of influence as a function of the well pumpage and the recharge rate only:

$$Q = \pi r_{\phi}^2 w$$

where: r_{ϕ} = radius of influence

A.14.3 Input Data

For the analysis of the radius of influence of a well, it is only necessary to define the constant discharge rate (Q) and accretion rate (w). The accretion rate is assumed constant in time and space. It may be estimated by averaging some portion (the estimated aquifer recharge rate) of the annual precipitation rate. For the analysis to consider the magnitude of drawdown, it is necessary to also input the aquifer transmissivity and initial saturated aquifer thickness.

A.14.4 Output Data

The Weber technique can predict the steady-state aquifer response (maximum drawdown and radius of influence) to a pumping well.

A.14.5 Limitation

The steady-state equation is based on the following assumptions:

- The aquifer is homogeneous, isotropic, and of uniform thickness.
- The discharge well penetrates and receives water through the entire thickness of the aquifer.

- The hydraulic conductivity is constant at all times and at all locations.

- Discharging and recharging have continued for a sufficient duration for the hydraulic system to reach steady state.

- Flow to the well is horizontal, radial, and laminar, and originates from a circular open water source with a fixed radius and elevation which surrounds the well.

- Recharge is uniform over the area.

- The rate of discharge from the well is constant and equals the accretion rate within the radius of influence.

This method of estimating accretion rate in semiarid to arid climates could result in significant apparent errors. Also, the equation becomes less accurate when evaluating the drawdown at some radius r in close proximity to the producing well.

A.14.6 Justification

The use of the Weber equation should present a more realistic (smaller) radius of influence than the use of the Theim-Forchheimer equation. The accuracy of the analysis is heavily dependent on the estimated accretion rate.

A.14.7 References:

Forchheimer, P., 1901. "Wasserbewegung durch Boden" Ver. deutcher Ing. Zeitschr, Vol. 45, pp 1736-1741, 1781-1788.

Theim, G., 1906. "Hydrologische Methoden" Leipzig, J.M. Gephardt, p 56.

Keck, W.G., 1976. Methods for estimating well yield, delivered at the National Water Well Association Annual Meeting, Las Vegas, Nevada.

APPENDIX B

APPENDIX B
HYDROGRAPH ROUTING PROGRAM - HYDROUT

B.1 Introduction

Program HYDROUT is written in FORTRAN and contains computerized routines for deriving basic hydrographs. It uses Soil Conservation Services (SCS) TR-55 methods to generate sub-basin hydrographs and then routes each hydrograph to a common point downstream in accordance with the associated travel time. This program was used to generate flood hydrographs for the Crow Creek Basin. This appendix contains the followings aspects of program documentation:

- o Input data requirements (B.2);
- o Program listing (B.3); and
- o Program output (B.4).

B.2 Input Data Requirements

Program HYDROUT requires the following groups of input data:

group 1: job description cards

group 2: job control ends

group 3: a nondimensional unit hydrograph which describes the shape of the hydrograph

group 4: sub-basin characteristics

Input descriptions are given in Table B.2-1. A sample input data file is shown in Table B.2-2.

Table B.2-1

INPUT DESCRIPTION

Group	Variable	Format	Description
1	TITLE	10A8	Job title
2	NB	I5	Number of sub-basins
	NP	I5	Number of points are used in describing the nondimensional hydrograph (see group 3)
	IFIN	I5	Number of time steps in the simulation
	FREQ	F10.0	Return frequency for the storm event
	D	F10.0	Duration of the storm event
	P	F10.0	Total precipitation (in inches) during the storm event
	TSTEP	F10.0	Time step (in minutes) used in the simulation
3	TPTP	8F10.0	A nondimensional factor, t/t_p, where t is time and t_p is time to peak (each card contains eight points, continue the input to next cards when NP is greater than 8)
	QPQP	8F10.0	A nondimensional factor, Q/Q_p, where Q is flow and Q_p is flow peak, together with TPTP, it describes the shape of the unit hydrograph
4	NBASIN	A8,2X	Sub-basin name
	A	F10.0	Sub-basin area, in acres
	TC	F10.0	Time of concentration, in hours
	TT	F10.0	Travel time from the inlet (at Crow Creek) to a prescribed point downstream
	CN	F10.0	Curve number

TABLE B.2-2
SAMPLE INPUT DATA FILE

CROW CREEK (INCLUDING CHEYENNE) FLOOD HYDROGRAPH - FOR 50 YR 6 HR STORM
1987 LANDUSE

50. Year 6. Hour 2.90 Inches 30. Minute Time Steps

T/T_p	0.	.1	.2	.3	.4	.5	.6	.7
	.8	.9	1.0	1.1	1.2	1.3	1.4	1.5
	1.6	1.7	1.8	1.9	2.0	2.2	2.4	2.6
	2.8	3.0	3.2	3.4	3.6	4.0	4.5	5.0
Q/Q_p	.000	.030	.100	.190	.310	.470	.660	.820
	.930	.990	1.000	.990	.930	.860	.780	.680
	.560	.460	.390	.330	.280	.207	.147	.107
	.077	.055	.040	.029	.021	.011	.005	.000

Basin Data	NC1	7.40	2.10	21.59	70.0
	NC2	8.67	4.77	13.08	70.0
	NC3	7.00	5.87	18.45	70.0
	NC4	4.44	6.95	19.40	70.0
	NC5	9.02	8.35	17.46	70.0
	NC6	3.63	3.06	16.27	70.0
	NC7	6.33	6.07	16.27	70.0
	NC8	26.00	19.65	12.50	70.0
	NC9	8.39	15.87	13.29	70.0
	MC1	10.31	7.03	23.24	70.0
	MC2	10.93	16.30	20.62	70.0
	MC3	9.33	10.79	17.66	70.0
	MC4	2.59	1.33	18.25	70.0
	MC5	.75	2.22	17.66	70.0
	MC6	5.06	10.95	18.81	70.0
	MC7	7.22	7.14	16.47	70.0
	MC8	10.65	9.95	13.49	70.0
	SC1	4.40	9.72	23.59	70.0
	SC2	5.38	14.90	20.57	70.0
	SC3	3.12	12.73	25.47	70.0
	SC4	2.53	6.09	22.46	70.0
	SC5	2.84	4.97	18.91	70.0
	SC6	1.43	3.87	17.49	70.0
	SC7	3.09	9.26	19.86	70.0
	SC8	4.90	4.10	18.91	70.0
	SC9	2.09	4.62	17.49	70.0
	SC10	3.71	4.93	16.08	70.0
	C1	.90	.62	19.74	70.0
	C2	4.70	2.37	18.68	70.0
	C3	6.05	13.40	14.39	70.0
	C4	3.75	7.92	14.89	70.0
	C5	7.94	16.66	13.48	70.0
	C6	12.95	23.84	11.47	70.0
	C7	9.73	29.54	10.76	70.0
	C8	6.90	14.87	11.47	70.0
	C9	5.04	14.62	11.47	70.0
	C10	3.28	12.40	9.51	70.0
	C11	5.50	13.37	9.51	70.0
	C12	10.89	29.24	9.51	70.0
	C13	4.25	11.11	9.46	70.0
	C14	9.51	28.59	9.46	70.0
	C15	10.28	17.41	6.62	71.5
	C16	7.57	11.53	6.62	72.3
	C17	10.83	20.33	6.62	72.4
	C18	8.73	10.19	6.38	71.1
	C19	2.00	7.44	5.44	76.7
	C20	0.74	7.44	5.44	81.8
	C21A	2.97	9.66	7.40	74.3
	C21B	5.07	11.11	5.41	74.9
	C21C	5.91	10.45	3.55	74.1
	C22	4.76	16.5	4.02	79.2
	C23	9.58	11.93	4.02	72.6
	C24	7.66	17.22	0.	70.1
	C25	5.32	11.51	0.	70.0
	Basin	Area (sq mi)	Time of Concentration	Travel Time (hrs)	Curve Number

B.3 Program Output

The flood hydrographs were developed for the following three storm events:

- o A 2-year, 24-hour storm; total rainfall = 1.68 in.;
- o A 10-year, 3-hour storm; total rainfall = 1.71 in.; and
- o A 50-year, 6-hour storm; total rainfall = 2.90 in.

and under three different land use scenarios:

- o Existing land use;
- o 1987 land use without the project; and
- o 1987 land use with the project.

Tables B.4-1 to B.4-12 list the associated program output data files. Each output data file consists of five separate sections:

1. Job title
2. Job control parameters
3. Nondimensional unit hydrograph
4. Sub-basin data files
5. Routed flood hydrograph

```
      PROGRAM HYDROUT(INPUT,OUTPUT,TAPE1,TAPE5=INPUT,TAPE6=OUTPUT)
C     THIS PROGRAM WILL DERIVE SUBBASIN HYDROGRAPHS IN ACCORDING
C     TO SCS TR-55 METHOD, USING THE FOLLOWING INPUT PARAMETERS,
C     AREA(A),TIME CONCENTRATION(TC),AND SHAPE OF THE HYDRO-
C     GRAPHS(Q/QP VS. T/TP)
C     EACH SUBBASIN HYDROGRAPH THEN ROUTED TO A COMMON DOWNSTREAM
C     POINT BY THE ASSOCIATED TRAVEL TIME(TT).
C
C     THIS VERSION IS UPDATED ON JAN, 1984
C
      COMMON /A/ TPTP(60),QPQP(60),Q(60,400),QP(60)
      COMMON /B/ T(60),QQ(60),QROUT(400),TITLE(20)
      COMMON /C/ DIST(60),PT(60),PINC(60),QRM1(60)
C
      READ(1,5) (TITLE(I),I=1,10)
      READ(1,5) (TITLE(I),I=11,20)
      WRITE(6,5) (TITLE(I),I=1,10)
      WRITE(6,5) (TITLE(I),I=11,20)
    5 FORMAT(10A8)
      WRITE(6,6)
    6 FORMAT(//)
      READ(1,1) NB,NP,IFIN,FREQ,D,P,TSTEP
      ID=D*60./TSTEP
      READ(1,10) (DIST(I),I=1,ID)
      READ(5,10) (TPTP(J),J=1,NP)
      READ(5,10) (QPQP(J),J=1,NP)
C
      WRITE(6,20) NB,NP,IFIN,FREQ,D,P,TSTEP
      WRITE(6,30) (TPTP(J),J=1,NP)
      WRITE(6,35) (QPQP(J),J=1,NP)
      WRITE(6,15)
C
C     ** INITIALIZE VARIABLES **
C
      PCUM=0.0
C
      DO 9 N=1,NB
      QRM1(N)=0.0
      DO 9 I=1,IFIN
      Q(N,I)=0.0
    8 CONTINUE
C
      DO 9 I=1,ID
      PINC(I)=P*DIST(I)
      PCUM=PCUM+PINC(I)
      PT(I)=PCUM
    9 CONTINUE
C
      DO 1000 N=1,NB
      READ(5,2) NBASIN,A,TC,TT,CN
      BQP=0.
C
      S=1000./CN-10.
      UTP=TSTEP/(2.*60.)+0.6*TC
      BTP=D/2+0.6*TC
      IN=TT*60./TSTEP
      IF(IN.LT.1) GO TO 45
      DO 40 I=1,IN
      Q(N,I)=0.
   40 CONTINUE
   45 IM=0
C
      DO 150 I=1,ID
      DIFF=PT(I)-0.2*S
      IF(DIFF.LE.0.) 46,47
   46 QR=0.0
      QP(I)=0.0
      GO TO 48
   47 QR=(PT(I)-0.2*S)**2./(PT(I)+0.3*S)
      QRINC=QR-QRM1(N)
      QRM1(N)=QR
      QP(I)=484.*A*QRINC/UTP
C
C     ** COMPUTE UNIT HYDROGRAPH **
C
      IT=0
   48 DO 100 J=1,NP
      ITEMP=IM
      JP1=J+1
      T(J)=TPTP(J)*UTP*60.+(I-1)*TSTEP
      IM=T(J)/TSTEP+1
      IDELT=IM-ITEMP
C
```

```
C     IDELT IS NUMBER OF TIME STEPS BETWEEN T(J) AND T(J+1)
C
      QQ(J)=QPQP(J)*QP(I)
      IF(JP1.GT.NP) GO TO 70
      QQ(JP1)=QPQP(JP1)*QP(I)
      IF(IDELT-1) 55,55,60
C
C     NO INTERPOLATION
C
   55 IF(J.GT.1) IT=T(J-1)/TSTEP+1
      IDIFF=IM-IT
      IF(IDIFF.NE.1) GO TO 100
      IK=IN+IM
      Q(N,IK)=Q(N,IK)+QQ(J)
      QTEMP=Q(N,IK)
      BQP=AMAX1(BQP,QTEMP)
      GO TO 100
C
C     WITH INTERPOLATION
C
   60 DO 65 IJ=1,IDELT
      IK=IN+ITEMP+IJ
      Q(N,IK)=Q(N,IK)+QQ(J)+(QQ(JP1)-QQ(J))*(IJ-1)/IDELT
      QTEMP=Q(N,IK)
      BQP=AMAX1(BQP,QTEMP)
   65 CONTINUE
C
      GO TO 100
   70 IF(IDELT-1) 72,72,74
   72 IK=IN+IM
      Q(N,IK)=Q(N,IK)+QQ(J)
      QTEMP=Q(N,IK)
      BQP=AMAX1(BQP,QTEMP)
      GO TO 100
   74 DO 76 IJ=1,IDELT
      IK=IN+ITEMP+IJ
      Q(N,IK)=Q(N,IK)+QQ(J)+(0.-QQ(J))*(IJ-1)/IDELT
      QTEMP=Q(N,IK)
      BQP=AMAX1(BQP,QTEMP)
   76 CONTINUE
  100 CONTINUE
  150 CONTINUE
      IM1=IK+1
      DO 200  II=IM1,IFIN
      Q(N,II)=0.
  200 CONTINUE
C
      WRITE(6,25) NBASIN,A,TC,TT,BTP,CN,BQP
   25 FORMAT(1X,A8,1X,6F10.2)
 1000 CONTINUE
C
      DO 2000 I=1,IFIN
      SUMQ=0.
      DO 1500 N=1,NB
      SUMQ=SUMQ+Q(N,I)
 1500 CONTINUE
C
      QROUT(I)=SUMQ
 2000 CONTINUE
C
      WRITE(6,49) TSTEP
      WRITE(6,50) (QROUT(I),I=1,IFIN)
C
    1 FORMAT(3I5,4F10.0)
    2 FORMAT(A8,2X,4F10.0)
   10 FORMAT(8F10.0)
   15 FORMAT(67H BASIN      AREA(SQ.M)   TC(HR)    TT(HR)    TP(HR)
     1N         QP)
   20 FORMAT(/,* NB =*,I5,2X,* NP =*,I5,2X,* IFIN =*,I5,/,
     1F5.0,* YR AND *,F5.0,* HR DURATION STORM =*,F8.2,* IN *,/,
     2* TIME STEP =*,F5.0,* MINUTES*)
   30 FORMAT(/,* T/TP =*,14F5.2/(7X,14F5.2))
   35 FORMAT(* Q/QP =*,14F5.2/(7X,14F5.2))
   49 FORMAT(* FLOOD HYDROGRAPH IN EVERY *,F5.0,* MINUTES *)
   50 FORMAT(12F6.0)
      STOP
      END
501 ENCOUNTERED.
```

CROW CREEK AT DIAMOND - FOR 2 YR 24 HR STORM
1993 LANDUSE

NB = 39 NP = 32 IFIN = 250
2. YR AND 24. HR DURATION STORM = 1.68 IN
TIME STEP = 30. MINUTES

T/TP = 0.00 .10 .20 .30 .40 .50 .60 .70 .80 .90 1.00 1.10 1.20 1.30
1.40 1.50 1.60 1.70 1.80 1.90 2.00 2.20 2.40 2.60 2.80 3.00 3.20 3.40
3.60 4.00 4.50 5.00
Q/QP = 0.00 .03 .10 .19 .31 .47 .66 .82 .93 .99 1.00 .99 .93 .86
.78 .68 .56 .46 .39 .33 .28 .21 .15 .11 .08 .06 .04 .03
.02 .01 .01 0.00

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	21.59	13.26	70.00	60.41
NC2	8.67	4.77	13.08	14.86	70.00	67.46
NC3	7.00	5.87	18.45	15.52	70.00	53.53
NC4	4.44	6.95	19.40	16.17	70.00	32.78
NC5	8.02	8.35	17.46	17.01	70.00	55.29
NC6	3.63	3.06	16.27	13.84	70.00	29.48
NC7	6.33	6.07	16.27	15.64	70.00	48.29
NC8	26.00	19.65	12.50	23.79	70.00	121.07
NC9	8.39	15.87	13.29	21.52	70.00	45.51
MC1	10.31	7.03	23.24	16.22	70.00	75.66
MC2	10.83	16.30	20.62	21.78	70.00	57.23
MC3	9.33	10.79	17.66	18.47	70.00	60.53
MC4	2.59	1.33	18.25	12.80	70.00	20.75
MC5	.75	2.22	17.66	13.33	70.00	6.26
MC6	5.06	10.95	18.81	18.57	70.00	32.92
MC7	7.22	7.14	16.47	16.28	70.00	52.89
MC8	10.65	9.95	13.49	17.97	70.00	71.94
SC1	4.40	9.72	23.59	17.93	70.00	29.95
SC2	5.88	14.90	20.57	20.94	70.00	32.70
SC3	3.12	12.73	25.47	19.64	70.00	18.93
SC4	2.53	6.09	22.46	15.65	70.00	19.20
SC5	2.84	4.97	18.91	14.98	70.00	22.21
SC6	1.43	3.87	17.49	14.32	70.00	11.22
SC7	3.09	9.26	19.36	17.56	70.00	21.42
SC8	4.90	4.10	18.91	14.46	70.00	38.80
SC9	2.09	4.62	17.49	14.77	70.00	16.36
SC10	3.71	4.93	16.08	14.96	70.00	28.82
C1	.90	.62	19.74	12.37	70.00	7.40
C2	4.70	2.37	18.68	13.42	70.00	38.05
C3	6.05	13.40	14.89	20.04	70.00	35.88
C4	3.75	7.92	14.89	16.75	70.00	26.95
C5	7.94	16.66	13.48	22.00	70.00	41.07
C6	12.95	23.84	11.47	26.30	70.00	51.98
C7	8.73	29.54	10.76	29.72	70.00	29.28
C8	6.90	14.87	11.47	20.92	70.00	38.45
C9	5.04	14.62	11.47	20.77	70.00	28.40
C10	3.28	12.40	8.51	19.44	70.00	20.11
C11	5.50	13.37	8.51	20.02	70.00	32.69
C12	10.89	29.24	8.51	29.54	70.00	36.94

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	1.	1.	2.	4.
6.	8.	11.	16.	23.	33.	45.	61.	80.	102.	129.	160.
196.	240.	290.	348.	411.	478.	547.	616.	685.	754.	822.	888.
951.	1008.	1058.	1104.	1144.	1180.	1208.	1229.	1243.	1250.	1248.	1236.
1217.	1189.	1153.	1111.	1058.	1004.	952.	903.	853.	800.	748.	698.
654.	613.	574.	536.	498.	463.	429.	396.	365.	336.	308.	284.
260.	239.	219.	201.	184.	169.	155.	143.	131.	120.	110.	101.
93.	86.	79.	73.	67.	62.	57.	53.	49.	46.	42.	39.
37.	34.	32.	29.	27.	26.	24.	22.	21.	19.	18.	17.
16.	15.	14.	13.	12.	11.	10.	10.	9.	9.	8.	7.
7.	6.	6.	6.	5.	5.	5.	4.	4.	4.	4.	3.
3.	3.	3.	3.	2.	2.	2.	2.	2.	2.	2.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.		

CROW CREEK AT DIAMOND - FOR 10YR 3 HR STORM
1983 LANDUSE

NB = 39 NP = 32 IFIN = 250
10. YR AND 3. HR DURATION STORM = 1.71 IN
TIME STEP = 30. MINUTES

T/TP =	0.00	.10	.20	.30	.40	.50	.60	.70	.80	.90	1.00	1.10	1.20	1.30
	1.40	1.50	1.60	1.70	1.80	1.90	2.00	2.20	2.40	2.60	2.80	3.00	3.20	3.40
	3.60	4.00	4.50	5.00										
Q/QP =	0.00	.03	.10	.19	.31	.47	.66	.82	.93	.99	1.00	.99	.93	.86
	.78	.68	.56	.46	.39	.33	.28	.21	.15	.11	.08	.06	.04	.03
	.02	.01	.01	0.00										

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	14.45	2.76	70.00	296.48
NC2	8.67	4.77	12.88	4.36	70.00	188.28
NC3	7.00	5.87	11.31	5.02	70.00	125.38
NC4	4.44	6.95	12.26	5.67	70.00	67.91
NC5	8.02	8.35	10.32	6.51	70.00	104.34
NC6	3.63	3.06	9.13	3.34	70.00	114.47
NC7	6.33	6.07	9.13	5.14	70.00	109.49
NC8	26.00	19.65	5.36	13.29	70.00	148.37
NC9	8.39	15.87	6.15	11.02	70.00	58.96
MC1	10.31	7.03	16.10	5.72	70.00	155.34
MC2	10.83	16.30	13.40	11.28	70.00	74.15
MC3	9.33	10.79	10.52	7.97	70.00	95.15
MC4	2.59	1.33	11.11	2.30	70.00	124.21
MC5	.75	2.22	10.52	2.83	70.00	29.22
MC6	5.06	10.95	11.67	8.07	70.00	50.89
MC7	7.22	7.14	9.33	5.78	70.00	107.62
MC8	10.65	9.95	6.35	7.47	70.00	117.41
SC1	4.40	9.72	15.08	7.33	70.00	49.62
SC2	5.38	14.90	12.06	10.44	70.00	43.94
SC3	3.12	12.73	17.23	9.14	70.00	27.13
SC4	2.53	6.09	13.95	5.15	70.00	43.63
SC5	2.34	4.97	10.40	4.48	70.00	58.60
SC6	1.43	3.87	8.98	3.82	70.00	36.60
SC7	3.09	9.26	11.35	7.06	70.00	36.42
SC8	4.90	4.10	10.40	3.96	70.00	119.11
SC9	2.09	4.62	8.98	4.27	70.00	46.71
SC10	3.71	4.93	7.57	4.46	70.00	77.12
C1	.90	.62	11.23	1.87	70.00	60.45
C2	4.70	2.37	10.17	2.92	70.00	174.58
C3	6.05	13.40	6.38	9.54	70.00	50.06
C4	3.75	7.92	6.38	6.25	70.00	51.30
C5	7.94	16.66	4.96	11.50	70.00	53.22
C6	12.95	23.84	2.96	15.80	70.00	61.17
C7	8.73	29.54	2.25	19.22	70.00	33.40
C8	6.90	14.87	2.96	10.42	70.00	51.65
C9	5.04	14.62	2.96	10.27	70.00	38.36
C10	3.28	12.40	0.00	8.94	70.00	29.25
C11	5.50	13.37	0.00	9.52	70.00	45.61
C12	10.89	29.24	0.00	19.04	70.00	42.09

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	1.	3.	8.	13.	19.	28.	40.	55.
72.	89.	108.	129.	154.	184.	223.	271.	328.	397.	485.	599.
756.	952.	1219.	1497.	1681.	1677.	1605.	1571.	1591.	1664.	1776.	1815.
1732.	1582.	1438.	1320.	1226.	1163.	1106.	1052.	998.	943.	887.	826.
766.	707.	651.	597.	544.	490.	442.	402.	370.	342.	315.	289.
263.	239.	217.	197.	180.	165.	151.	138.	127.	117.	107.	98.
90.	83.	77.	71.	65.	60.	56.	52.	48.	45.	42.	39.
36.	34.	32.	29.	27.	26.	24.	23.	21.	20.	18.	17.
16.	15.	14.	13.	12.	11.	10.	10.	9.	8.	8.	7.
7.	6.	6.	6.	5.	5.	5.	4.	4.	4.	3.	3.
3.	3.	3.	2.	2.	2.	2.	2.	2.	2.	2.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.		

CROW CREEK AT JIAMOND - FOR 50 YR 6 HR DTORM
1983 LANDUSE

NB = 39 NP = 32 IFIN = 250
50. YR AND 6. HR DURATION STORM = 2.90 IN
TIME STEP = 30. MINUTES

T/TP = 0.00 .10 .20 .30 .40 .50 .60 .70 .80 .90 1.00 1.10 1.20 1.30
1.40 1.50 1.60 1.70 1.80 1.90 2.00 2.20 2.40 2.60 2.80 3.00 3.20 3.40
3.60 4.00 4.50 5.00
Q/QP = 0.00 .03 .10 .19 .31 .47 .66 .82 .93 .99 1.00 .99 .93 .86
.78 .68 .56 .46 .39 .33 .28 .21 .15 .11 .08 .06 .04 .03
.02 .01 .01 0.00

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	14.45	4.26	70.00	858.83
NC2	8.67	4.77	12.88	5.86	70.00	728.09
NC3	7.00	5.87	11.31	6.52	70.00	516.01
NC4	4.44	6.95	12.26	7.17	70.00	287.50
NC5	8.02	8.35	10.32	8.01	70.00	450.98
NC6	3.63	3.06	9.13	4.84	70.00	379.15
NC7	6.33	6.07	9.13	6.64	70.00	456.25
NC8	26.00	19.65	5.36	14.79	70.00	681.24
NC9	8.39	15.87	6.15	12.52	70.00	269.07
MC1	10.31	7.03	16.10	7.22	70.00	659.32
MC2	10.83	16.30	13.40	12.78	70.00	338.39
MC3	9.33	10.79	10.52	9.47	70.00	425.06
MC4	2.59	1.33	11.11	3.80	70.00	306.01
MC5	.75	2.22	10.52	4.38	70.00	88.27
MC6	5.06	10.95	11.67	9.57	70.00	228.82
MC7	7.22	7.14	9.33	7.28	70.00	455.80
MC8	10.65	9.95	6.35	8.97	70.00	521.58
SC1	4.40	9.72	15.08	8.83	70.00	219.80
SC2	5.88	14.90	12.06	11.94	70.00	200.52
SC3	3.12	12.73	17.23	10.64	70.00	122.02
SC4	2.53	6.09	13.95	6.65	70.00	181.79
SC5	2.84	4.97	10.40	5.98	70.00	235.71
SC6	1.43	3.87	8.98	5.32	70.00	133.07
SC7	3.09	9.26	11.35	8.56	70.00	159.24
SC8	4.90	4.10	10.40	5.46	70.00	452.32
SC9	2.09	4.62	8.98	5.77	70.00	177.46
SC10	3.71	4.93	7.57	5.96	70.00	306.17
C1	.90	.62	11.23	3.37	70.00	111.70
C2	4.70	2.37	10.17	4.42	70.00	525.12
C3	6.05	13.40	6.38	11.04	70.00	226.51
C4	3.75	7.92	6.38	7.75	70.00	221.75
C5	7.94	16.66	4.96	13.00	70.00	242.86
C6	12.95	23.84	2.96	17.30	70.00	281.95
C7	8.73	29.54	2.25	20.72	70.00	154.16
C8	6.90	14.87	2.96	11.92	70.00	235.76
C9	5.04	14.62	2.96	11.77	70.00	174.20
C10	3.28	12.40	0.00	10.44	70.00	131.75
C11	5.50	13.37	0.00	11.02	70.00	206.36
C12	10.89	29.24	0.00	20.54	70.00	194.25

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	0.	1.	4.	10.	19.	33.	53.	81.
120.	166.	221.	286.	362.	449.	549.	664.	799.	965.	1163.	1412.
1727.	2127.	2639.	3300.	4070.	4864.	5652.	6346.	6910.	7320.	7609.	7687.
7687.	7721.	7741.	7660.	7493.	7180.	6719.	6213.	5769.	5401.	5084.	4814.
4553.	4293.	4029.	3783.	3499.	3232.	2972.	2722.	2486.	2285.	2063.	1878.
1712.	1568.	1439.	1319.	1207.	1101.	1002.	913.	832.	760.	695.	638.
585.	537.	493.	453.	417.	383.	353.	326.	301.	279.	258.	239.
222.	207.	192.	179.	167.	156.	145.	135.	126.	118.	110.	102.
97.	91.	85.	79.	74.	69.	64.	60.	55.	51.	48.	44.
41.	39.	36.	34.	32.	30.	28.	26.	24.	22.	21.	20.
18.	17.	16.	15.	14.	13.	12.	11.	11.	10.	9.	9.
8.	8.	7.	7.	6.	6.	5.	5.	4.	4.	4.	3.
3.	3.	3.	3.	3.	2.	2.	2.	2.	2.	2.	2.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.		

TABLE B.4-4

CROW CREEK (INCLUDING CHEYENNE FLOOD HYDROGRAPH - FOR 2 YR 24 HR STORM
1993 LANDUSE

NB = 54 NP = 32 IFIN = 250
2. YR AND 24. HR DURATION STORM = 1.68 IN
TIME STEP = 30. MINUTES

T/TP = 0.00 .10 .20 .30 .40 .50 .60 .70 .80 .90 1.00 1.10 1.20 1.30
1.40 1.50 1.60 1.70 1.80 1.90 2.00 2.20 2.40 2.60 2.80 3.00 3.20 3.40
3.60 4.00 4.50 5.00
Q/QP = 0.00 .03 .10 .19 .31 .47 .66 .82 .93 .99 1.00 .99 .93 .86
.78 .68 .56 .46 .39 .33 .28 .21 .15 .11 .08 .06 .04 .03
.02 .01 .01 0.00

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	21.59	13.26	70.00	60.41
NC2	8.67	4.77	13.08	14.86	70.00	67.46
NC3	7.00	5.87	18.45	15.52	70.00	53.53
NC4	4.44	6.95	19.40	16.17	70.00	32.78
NC5	8.02	8.35	17.46	17.01	70.00	55.29
NC6	3.63	3.06	16.27	13.84	70.00	29.48
NC7	6.33	6.07	16.27	15.64	70.00	48.20
NC8	26.00	19.65	12.50	23.79	70.00	121.07
NC9	8.39	15.87	13.29	21.52	70.00	45.51
MC1	10.31	7.03	23.24	16.22	70.00	75.66
MC2	10.83	16.30	20.62	21.78	70.00	57.23
MC3	9.33	10.79	17.66	18.47	70.00	60.53
MC4	2.59	1.33	18.25	12.80	70.00	20.75
MC5	.75	2.22	17.66	13.33	70.00	6.26
MC6	5.06	10.95	18.81	18.57	70.00	32.92
MC7	7.22	7.14	16.47	16.28	70.00	52.89
MC8	10.65	9.95	13.49	17.97	70.00	71.94
SC1	4.40	9.72	23.59	17.83	70.00	29.95
SC2	5.88	14.90	20.57	20.94	70.00	32.70
SC3	3.12	12.73	25.47	19.64	70.00	18.93
SC4	2.53	6.09	22.46	15.65	70.00	19.20
SC5	2.84	4.97	18.91	14.98	70.00	22.21
SC6	1.43	3.87	17.49	14.32	70.00	11.22
SC7	3.09	9.26	19.86	17.56	70.00	21.42
SC8	4.90	4.10	18.91	14.46	70.00	38.30
SC9	2.09	4.62	17.49	14.77	70.00	16.36
SC10	3.71	4.93	16.08	14.96	70.00	29.82
C1	.90	.62	19.74	12.37	70.00	7.40
C2	4.70	2.37	18.68	13.42	70.00	38.05
C3	6.05	13.40	14.89	20.04	70.00	35.88
C4	3.75	7.92	14.89	16.75	70.00	26.95
C5	7.94	16.66	13.48	22.00	70.00	41.07
C6	12.95	23.84	11.47	26.30	70.00	51.98
C7	8.73	29.54	10.76	29.72	70.00	29.28
C8	6.90	14.87	11.47	20.92	70.00	38.45
C9	5.04	14.62	11.47	20.77	70.00	28.40
C10	3.28	12.40	8.51	19.44	70.00	20.11
C11	5.50	13.37	8.51	20.02	70.00	32.69
C12	10.89	29.24	8.51	29.54	70.00	36.94
C13	4.25	11.11	9.46	18.67	70.00	27.39
C14	9.51	28.59	9.46	29.15	70.00	32.87
C15	10.28	17.41	6.62	22.45	71.50	62.18
C16	7.57	11.53	6.62	18.92	72.20	60.62
C17	10.83	20.83	6.62	24.50	72.40	64.18
C18	8.73	10.19	6.38	18.11	71.10	65.80
C19	2.00	7.44	5.44	16.46	76.30	26.37
C20	.74	7.44	5.44	16.46	81.60	14.78
C21A	2.97	9.66	7.40	17.80	74.10	30.00
C21B	5.07	11.11	5.41	18.67	74.60	51.79
C21C	5.91	10.45	3.55	18.27	73.90	57.80
C22	4.76	16.50	4.02	21.90	78.90	59.22
C23	9.58	11.93	4.02	19.16	72.50	78.80
C24	7.66	17.22	0.00	22.33	70.00	39.31
C25	5.82	10.75	0.00	18.45	70.83	41.52

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	1.	2.	4.	6.	9.	14.	20.	28.	38.
50.	65.	82.	103.	125.	150.	177.	206.	236.	268.	300.	333.
367.	400.	434.	467.	502.	537.	572.	609.	646.	684.	722.	763.
805.	851.	901.	953.	1007.	1061.	1115.	1166.	1215.	1261.	1305.	1347.
1385.	1417.	1443.	1464.	1482.	1496.	1503.	1503.	1499.	1488.	1470.	1444.
1410.	1369.	1322.	1268.	1205.	1141.	1079.	1022.	965.	904.	845.	789.
739.	692.	648.	605.	563.	523.	485.	449.	415.	382.	352.	324.
299.	275.	253.	232.	214.	197.	181.	167.	154.	141.	130.	120.
111.	102.	94.	87.	81.	75.	69.	64.	60.	55.	52.	48.
45.	42.	39.	36.	34.	31.	29.	27.	26.	24.	22.	21.
20.	18.	17.	16.	15.	14.	13.	12.	12.	11.	10.	9.
9.	8.	8.	7.	7.	6.	6.	6.	5.	5.	5.	4.
4.	4.	4.	3.	3.	3.	3.	3.	2.	2.	2.	2.
2.	2.	2.	2.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.		

CROW CREEK (INCLUDING CHEYENNE) FLOOD HYDROGRAPH - FOR 10 YR 3 HR STORM
1993 LANDUSE

TABLE B.4-5

NB = 54 NP = 32 IFIN = 250
10. YR AND 3. HR DURATION STORM = 1.71 IN
TIME STEP = 30. MINUTES

T/TP = 0.00 .10 .20 .30 .40 .50 .60 .70 .80 .90 1.00 1.10 1.20 1.30
1.40 1.50 1.60 1.70 1.80 1.90 2.00 2.20 2.40 2.60 2.80 3.00 3.20 3.40
3.60 4.00 4.50 5.00
Q/QP = 0.00 .03 .10 .19 .31 .47 .66 .82 .93 .99 1.00 .99 .93 .86
.78 .68 .56 .46 .39 .33 .28 .21 .15 .11 .08 .06 .04 .03
.02 .01 .01 0.00

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	21.59	2.76	70.00	296.48
NC2	8.67	4.77	13.08	4.36	70.00	188.28
NC3	7.00	5.87	18.45	5.02	70.00	125.38
NC4	4.44	6.95	19.40	5.67	70.00	67.91
NC5	8.02	8.35	17.46	6.51	70.00	104.34
NC6	3.63	3.06	16.27	3.34	70.00	114.47
NC7	6.33	6.07	16.27	5.14	70.00	109.49
NC8	26.00	19.65	12.50	13.29	70.00	148.37
NC9	8.39	15.87	13.29	11.02	70.00	58.96
MC1	10.31	7.03	23.24	5.72	70.00	155.34
MC2	10.83	16.30	20.62	11.28	70.00	74.15
MC3	9.33	10.79	17.66	7.97	70.00	95.15
MC4	2.59	1.33	18.25	2.30	70.00	124.21
MC5	.75	2.22	17.66	2.83	70.00	29.22
MC6	5.06	10.95	18.81	8.07	70.00	50.89
MC7	7.22	7.14	16.47	5.78	70.00	107.62
MC8	10.65	9.95	13.49	7.47	70.00	117.41
SC1	4.40	9.72	23.59	7.33	70.00	49.62
SC2	5.88	14.90	20.57	10.44	70.00	43.94
SC3	3.12	12.73	25.47	9.14	70.00	27.13
SC4	2.53	6.09	22.46	5.15	70.00	43.63
SC5	2.84	4.97	18.91	4.48	70.00	58.60
SC6	1.43	3.87	17.49	3.82	70.00	36.60
SC7	3.09	9.26	19.86	7.06	70.00	36.42
SC8	4.90	4.10	18.91	3.96	70.00	119.11
SC9	2.09	4.62	17.49	4.27	70.00	46.71
SC10	3.71	4.93	16.08	4.46	70.00	77.12
C1	.90	.62	19.74	1.87	70.00	60.45
C2	4.70	2.37	18.68	2.92	70.00	174.58
C3	6.05	13.40	14.89	9.54	70.00	50.06
C4	3.75	7.92	14.89	6.25	70.00	51.30
C5	7.94	16.66	13.48	11.50	70.00	53.22
C6	12.95	23.84	11.47	15.80	70.00	61.17
C7	8.73	29.54	10.76	19.22	70.00	33.40
C8	6.90	14.87	11.47	10.42	70.00	51.66
C9	5.04	14.62	11.47	10.27	70.00	38.36
C10	3.28	12.40	8.51	8.94	70.00	29.25
C11	5.50	13.37	8.51	9.52	70.00	45.61
C12	10.89	29.24	8.51	19.04	70.00	42.09
C13	4.25	11.11	9.46	8.17	70.00	42.06
C14	9.51	28.59	9.46	18.65	70.00	37.58
C15	10.28	17.41	6.62	11.95	71.50	79.26
C16	7.57	11.53	6.62	8.42	72.20	94.24
C17	10.33	20.33	6.62	14.00	72.40	77.72
C18	8.73	10.19	6.38	7.61	71.10	107.50
C19	2.00	7.44	5.44	5.96	76.30	57.54
C20	.74	7.44	5.44	5.96	81.60	33.73
C21A	2.97	9.66	7.40	7.30	74.10	53.88
C21B	5.07	11.11	5.41	8.17	74.60	84.38
C21C	5.91	10.45	3.55	7.77	73.90	97.36
C22	4.76	16.50	4.02	11.40	78.90	81.39
C23	9.58	11.93	4.02	8.66	72.50	119.31
C24	7.66	17.22	0.00	11.83	70.00	49.74
C25	5.82	10.75	0.00	7.95	70.83	66.00

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	1.	4.	9.	16.	23.	31.	42.	57.
74.	97.	130.	171.	215.	268.	335.	414.	492.	567.	636.	700.
764.	814.	851.	880.	902.	919.	933.	952.	985.	1031.	1081.	1132.
1165.	1205.	1278.	1368.	1481.	1631.	1802.	1957.	2061.	2096.	2020.	1937.
1910.	1931.	1910.	1791.	1630.	1488.	1388.	1318.	1265.	1215.	1158.	1101.
1043.	979.	913.	849.	788.	732.	675.	619.	567.	522.	481.	444.
410.	378.	349.	321.	294.	268.	245.	224.	207.	191.	175.	161.
149.	137.	126.	116.	108.	101.	93.	86.	79.	73.	68.	64.
59.	55.	51.	48.	45.	42.	39.	36.	34.	32.	30.	28.
26.	25.	23.	21.	20.	19.	17.	16.	15.	14.	13.	12.
12.	11.	10.	10.	9.	9.	8.	7.	7.	7.	6.	6.
5.	5.	5.	4.	4.	4.	4.	3.	3.	3.	3.	3.
3.	2.	2.	2.	2.	2.	2.	2.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.		

CROW CREEK (INCLUDING CHEYENNE) FLOOD HYDROGRAPH - FOR 50 YR 6 HR STORM
1983 LANDUSE

TABLE B.4-6

NB = 54 NP = 32 IFIN = 250
50. YR AND 6. HR DURATION STORM = 2.90 IN
TIME STEP = 30. MINUTES

T/TP = 0.00 .10 .20 .30 .40 .50 .60 .70 .80 .90 1.00 1.10 1.20 1.30
1.40 1.50 1.60 1.70 1.80 1.90 2.00 2.20 2.40 2.60 2.80 3.00 3.20 3.40
3.60 4.00 4.50 5.00
Q/QP = 0.00 .03 .10 .19 .31 .47 .66 .82 .93 .99 1.00 .99 .93 .86
.78 .68 .56 .46 .39 .33 .28 .21 .15 .11 .08 .06 .04 .03
.02 .01 .01 0.00

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	21.59	4.26	70.00	858.83
NC2	8.67	4.77	13.08	5.86	70.00	728.09
NC3	7.00	5.87	18.45	6.52	70.00	516.01
NC4	4.44	6.95	19.40	7.17	70.00	287.50
NC5	8.02	8.35	17.46	8.01	70.00	450.98
NC6	3.63	3.06	16.27	4.84	70.00	379.15
NC7	6.33	6.07	16.27	6.64	70.00	456.25
NC8	26.00	19.65	12.50	14.79	70.00	681.34
NC9	8.39	15.87	13.29	12.52	70.00	269.07
MC1	10.31	7.03	23.24	7.22	70.00	659.32
MC2	10.83	16.30	20.62	12.78	70.00	338.39
MC3	9.33	10.79	17.66	9.47	70.00	425.06
MC4	2.59	1.33	18.25	3.80	70.00	306.01
MC5	.75	2.22	17.66	4.33	70.00	88.27
MC6	5.06	10.95	18.81	9.57	70.00	228.92
MC7	7.22	7.14	16.47	7.28	70.00	455.80
MC8	10.65	9.95	13.49	8.97	70.00	521.58
SC1	4.40	9.72	23.59	8.83	70.00	219.80
SC2	5.88	14.90	20.57	11.94	70.00	200.52
SC3	3.12	12.73	25.47	10.64	70.00	122.02
SC4	2.53	6.09	22.46	6.65	70.00	181.79
SC5	2.84	4.97	18.91	5.98	70.00	235.71
SC6	1.43	3.87	17.49	5.32	70.00	133.07
SC7	3.09	9.26	19.86	8.56	70.00	159.24
SC8	4.90	4.10	18.91	5.46	70.00	452.32
SC9	2.09	4.62	17.49	5.77	70.00	177.46
SC10	3.71	4.93	16.08	5.96	70.00	306.17
C1	.90	.62	19.74	3.37	70.00	111.70
C2	4.70	2.37	18.68	4.42	70.00	525.12
C3	6.05	13.40	14.89	11.04	70.00	226.51
C4	3.75	7.92	14.89	7.75	70.00	221.75
C5	7.94	16.66	13.48	13.00	70.00	242.86
C6	12.95	23.84	11.47	17.30	70.00	281.95
C7	8.73	29.54	10.76	20.72	70.00	154.16
C8	6.90	14.87	11.47	11.92	70.00	235.76
C9	5.04	14.62	11.47	11.77	70.00	174.20
C10	3.28	12.40	8.51	10.44	70.00	131.75
C11	5.50	13.37	8.51	11.02	70.00	206.36
C12	10.89	29.24	8.51	20.54	70.00	194.25
C13	4.25	11.11	9.46	9.67	70.00	188.56
C14	9.51	28.59	9.46	20.15	70.00	173.43
C15	10.28	17.41	6.62	13.45	71.50	331.80
C16	7.57	11.53	6.62	9.92	72.20	370.67
C17	10.83	20.83	6.62	15.50	72.40	312.19
C18	8.73	10.19	6.38	9.11	71.10	451.20
C19	2.00	7.44	5.44	7.46	76.30	180.71
C20	.74	7.44	5.44	7.46	81.60	86.92
C21A	2.97	9.66	7.40	8.80	74.10	190.29
C21B	5.07	11.11	5.41	9.67	74.60	296.43
C21C	5.91	10.45	3.55	9.27	73.90	349.74
C22	4.76	16.50	4.02	12.90	78.90	245.78
C23	9.58	11.93	4.02	10.16	72.50	465.74
C24	7.66	17.22	0.00	13.33	70.00	228.03
C25	5.82	10.75	0.00	9.45	70.83	290.43

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	0.	1.	3.	11.	22.	37.	59.	88.
126.	173.	234.	308.	399.	511.	648.	810.	1001.	1219.	1456.	1706.
1971.	2243.	2499.	2738.	2954.	3142.	3311.	3452.	3578.	3702.	3837.	3996.
4169.	4361.	4591.	4851.	5133.	5482.	5884.	6348.	6908.	7497.	8029.	8474.
8821.	9025.	9068.	8996.	8754.	8460.	8141.	7810.	7422.	6948.	6475.	6074.
5757.	5482.	5224.	4965.	4691.	4413.	4136.	3853.	3576.	3311.	3059.	2821.
2598.	2391.	2198.	2026.	1869.	1721.	1584.	1457.	1337.	1226.	1124.	1030.
945.	868.	800.	736.	678.	625.	577.	533.	493.	456.	422.	392.
363.	336.	311.	289.	269.	251.	233.	218.	203.	190.	178.	166.
156.	146.	137.	128.	120.	112.	105.	98.	92.	85.	80.	74.
69.	65.	61.	57.	53.	50.	47.	44.	42.	39.	36.	34.
32.	30.	28.	26.	24.	23.	21.	20.	19.	18.	17.	16.
15.	14.	13.	13.	12.	11.	10.	10.	9.	8.	8.	7.
6.	6.	6.	5.	5.	4.	4.	4.	4.	3.	3.	3.
3.	3.	3.	2.	2.	2.	2.	2.	2.	1.	1.	1.
1.	1.	1.	1.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.		

CROW CREEK (INCLUDING CHEYENNE) FLOOD HYDROGRAPH - FOR 2 YR 24 HR STORM
1987 LANDUSE

TABLE B.4-7

NB = 54 NP = 32 IFIN = 250
2. YR AND 24. HR DURATION STORM = 1.68 IN
TIME STEP = 30. MINUTES

T/TP = 0.00 .10 .20 .30 .40 .50 .60 .70 .80 .90 1.00 1.10 1.20 1.30
1.40 1.50 1.60 1.70 1.80 1.90 2.00 2.20 2.40 2.60 2.80 3.00 3.20 3.40
3.60 4.00 4.50 5.00

Q/QP = 0.00 .03 .10 .19 .31 .47 .66 .82 .93 .99 1.00 .99 .93 .86
.73 .68 .56 .46 .39 .33 .28 .21 .15 .11 .08 .06 .04 .03
.02 .01 .01 0.00

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	21.59	13.26	70.00	60.41
NC2	8.67	4.77	13.08	14.86	70.00	67.46
NC3	7.00	5.87	18.45	15.52	70.00	53.53
NC4	4.44	6.95	19.40	16.17	70.00	32.78
NC5	8.02	8.35	17.46	17.01	70.00	55.29
NC6	3.63	3.06	16.27	13.84	70.00	29.48
NC7	6.33	6.07	16.27	15.64	70.00	48.20
NC8	26.00	19.65	12.50	23.79	70.00	121.07
NC9	8.39	15.87	13.29	21.52	70.00	45.51
MC1	10.31	7.03	23.24	16.22	70.00	75.66
MC2	10.83	16.30	20.62	21.78	70.00	57.23
MC3	9.33	10.79	17.66	18.47	70.00	60.53
MC4	2.59	1.33	18.25	12.80	70.00	20.75
MC5	.75	2.22	17.66	13.33	70.00	6.26
MC6	5.06	10.95	18.81	18.57	70.00	32.92
MC7	7.22	7.14	16.47	16.28	70.00	52.89
MC8	10.65	9.95	13.49	17.97	70.00	71.94
SC1	4.40	9.72	23.59	17.83	70.00	29.95
SC2	5.98	14.90	20.57	20.94	70.00	32.70
SC3	3.12	12.73	25.47	19.64	70.00	18.93
SC4	2.53	6.09	22.46	15.65	70.00	19.20
SC5	2.84	4.97	18.91	14.98	70.00	22.21
SC6	1.43	3.87	17.49	14.32	70.00	11.22
SC7	3.09	9.26	19.86	17.56	70.00	21.42
SC8	4.90	4.10	18.91	14.46	70.00	38.80
SC9	2.09	4.62	17.49	14.77	70.00	16.36
SC10	3.71	4.93	16.08	14.96	70.00	28.82
C1	.90	.62	19.74	12.37	70.00	7.40
C2	4.70	2.37	18.68	13.42	70.00	38.05
C3	6.05	13.40	14.89	20.04	70.00	35.88
C4	3.75	7.92	14.89	16.75	70.00	26.95
C5	7.94	16.66	13.48	22.00	70.00	41.07
C6	12.95	23.84	11.47	26.30	70.00	51.98
C7	8.73	29.54	10.76	29.72	70.00	29.28
C8	6.90	14.87	11.47	20.92	70.00	38.45
C9	5.04	14.62	11.47	20.77	70.00	28.40
C10	3.28	12.40	8.51	19.44	70.00	20.11
C11	5.50	13.37	8.51	20.02	70.00	32.69
C12	10.89	29.24	8.51	29.54	70.00	36.94
C13	4.25	11.11	9.46	18.67	70.00	27.39
C14	9.51	28.59	9.46	29.15	70.00	32.87
C15	10.28	17.41	6.62	22.45	71.55	62.53
C16	7.57	11.50	6.62	18.92	72.28	61.11
C17	10.83	20.83	6.62	24.50	72.39	64.11
C18	8.73	10.19	6.38	18.11	71.09	65.73
C19	2.00	7.44	5.44	16.46	76.65	27.12
C20	.74	7.44	5.44	16.46	81.84	15.06
C21A	2.97	9.66	7.40	17.80	74.25	30.41
C21B	5.07	11.11	5.41	18.67	74.87	53.06
C21C	5.91	10.45	3.55	18.27	74.01	58.37
C22	4.76	16.50	4.02	21.90	79.20	60.68
C23	9.58	11.93	4.02	19.16	72.61	79.66
C24	7.66	17.22	0.00	22.33	70.13	39.92
C25	5.82	10.75	0.00	18.45	70.83	41.52

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	1.	2.	4.	6.	10.	15.	21.	29.	39.
52.	67.	85.	106.	129.	154.	182.	211.	242.	274.	306.	340.
374.	407.	441.	475.	509.	544.	579.	616.	653.	691.	730.	770.
812.	858.	907.	959.	1012.	1067.	1120.	1170.	1219.	1265.	1309.	1350.
1388.	1420.	1445.	1467.	1484.	1498.	1504.	1505.	1500.	1490.	1472.	1445.
1411.	1370.	1322.	1269.	1205.	1141.	1080.	1022.	965.	905.	845.	790.
739.	692.	648.	605.	564.	523.	486.	449.	415.	382.	352.	325.
299.	275.	253.	233.	214.	197.	181.	167.	154.	141.	130.	120.
111.	102.	94.	87.	81.	75.	69.	64.	60.	55.	52.	48.
45.	42.	39.	36.	34.	31.	29.	27.	26.	24.	22.	21.
20.	18.	17.	16.	15.	14.	13.	12.	12.	11.	10.	9.
9.	8.	8.	7.	7.	6.	6.	6.	5.	5.	5.	4.
4.	4.	4.	3.	3.	3.	3.	3.	2.	2.	2.	2.
2.	2.	2.	2.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.		

CROW CREEK (INCLUDING CHEYENNE) FLOOD HYDROGRAPH - FOR 10 YR 3 HR STORM
1987 LANDUSE

TABLE B.4-8

NB = 54 NP = 32 IFIN = 250
10. YR AND 3. HR DURATION STORM = 1.71 IN
TIME STEP = 30. MINUTES

T/TP = 0.00 .10 .20 .30 .40 .50 .60 .70 .80 .90 1.00 1.10 1.20 1.30
1.40 1.50 1.60 1.70 1.80 1.90 2.00 2.20 2.40 2.60 2.80 3.00 3.20 3.40
3.60 4.00 4.50 5.00
Q/QP = 0.00 .03 .10 .19 .31 .47 .66 .82 .93 .99 1.00 .99 .93 .86
.78 .68 .56 .46 .39 .33 .28 .21 .15 .11 .08 .06 .04 .03
.02 .01 .01 0.00

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	21.59	2.76	70.00	296.48
NC2	8.67	4.77	13.08	4.36	70.00	189.28
NC3	7.00	5.87	18.45	5.02	70.00	125.38
NC4	4.44	6.95	19.40	5.67	70.00	67.91
NC5	8.02	8.35	17.46	6.51	70.00	104.34
NC6	3.63	3.06	16.27	3.34	70.00	114.47
NC7	6.33	6.07	16.27	5.14	70.00	109.49
NC8	26.00	19.65	12.50	13.29	70.00	148.37
NC9	8.39	15.87	13.29	11.02	70.00	58.96
MC1	10.31	7.03	23.24	5.72	70.00	155.34
MC2	10.83	16.30	20.62	11.28	70.00	74.15
MC3	9.33	10.79	17.66	7.97	70.00	95.15
MC4	2.59	1.33	18.25	2.30	70.00	124.21
MC5	.75	2.22	17.66	2.83	70.00	29.22
MC6	5.06	10.95	18.81	8.07	70.00	50.89
MC7	7.22	7.14	16.47	5.78	70.00	107.62
MC8	10.65	9.95	13.49	7.47	70.00	117.41
SC1	4.40	9.72	23.59	7.33	70.00	49.62
SC2	5.88	14.90	20.57	10.44	70.00	43.04
SC3	3.12	12.73	25.47	9.14	70.00	27.13
SC4	2.53	6.09	22.46	5.15	70.00	43.63
SC5	2.84	4.97	18.91	4.48	70.00	58.60
SC6	1.43	3.87	17.49	3.82	70.00	36.60
SC7	3.09	9.26	19.86	7.06	70.00	36.42
SC8	4.90	4.10	18.91	3.96	70.00	119.11
SC9	2.09	4.62	17.49	4.27	70.00	46.71
SC10	3.71	4.93	16.08	4.46	70.00	77.12
C1	.90	.62	19.74	1.87	70.00	60.45
C2	4.70	2.37	18.68	2.92	70.00	174.58
C3	6.05	13.40	14.89	9.54	70.00	50.06
C4	3.75	7.92	14.89	6.25	70.00	51.30
C5	7.94	16.66	13.48	11.50	70.00	53.22
C6	12.95	23.84	11.47	15.80	70.00	61.17
C7	8.73	29.54	10.76	19.22	70.00	33.40
C8	6.90	14.87	11.47	10.42	70.00	51.76
C9	5.04	14.62	11.47	10.27	70.00	38.36
C10	3.28	12.40	8.51	8.94	70.00	29.25
C11	5.50	13.37	8.51	9.52	70.00	45.61
C12	10.89	29.24	8.51	19.04	70.00	42.09
C13	4.25	11.11	9.46	8.17	70.00	42.06
C14	9.51	28.59	9.46	18.65	70.00	37.58
C15	10.28	17.41	6.62	11.95	71.55	79.73
C16	7.57	11.53	6.62	8.42	72.28	95.09
C17	10.83	20.83	6.62	14.00	72.39	77.63
C18	8.73	10.19	6.38	7.61	71.09	107.37
C19	2.00	7.44	5.44	5.96	76.65	59.47
C20	.74	7.44	5.44	5.96	81.84	34.38
C21A	2.97	9.66	7.40	7.30	74.25	54.73
C21B	5.07	11.11	5.41	8.17	74.87	86.71
C21C	5.91	10.45	3.55	7.77	74.01	98.49
C22	4.76	16.50	4.02	11.40	79.20	83.55
C23	9.58	11.93	4.02	8.66	72.61	120.78
C24	7.66	17.22	0.00	11.83	70.13	50.57
C25	5.82	10.75	0.00	7.95	70.83	66.00

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	1.	4.	9.	16.	23.	31.	42.	57.
75.	98.	132.	173.	219.	272.	340.	421.	500.	576.	646.	711.
775.	825.	862.	891.	913.	930.	943.	961.	993.	1039.	1088.	1139.
1172.	1211.	1283.	1373.	1486.	1635.	1805.	1960.	2063.	2099.	2022.	1939.
1911.	1932.	1911.	1792.	1631.	1489.	1389.	1319.	1266.	1215.	1159.	1102.
1044.	979.	914.	850.	788.	732.	675.	620.	567.	522.	482.	444.
410.	379.	349.	321.	294.	268.	245.	224.	207.	191.	175.	162.
149.	137.	126.	116.	108.	101.	93.	86.	79.	73.	68.	64.
59.	55.	51.	48.	45.	42.	39.	36.	34.	32.	30.	28.
26.	25.	23.	21.	20.	19.	17.	16.	15.	14.	13.	12.
12.	11.	10.	10.	9.	9.	8.	7.	7.	7.	6.	6.
5.	5.	5.	4.	4.	4.	4.	3.	3.	3.	3.	3.
3.	2.	2.	2.	2.	2.	2.	2.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.		

CROW CREEK (INCLUDING CHEYENNE) FLOOD HYDROGRAPH - FOR 50 YR 6 HR STORM **TABLE B.4-9**
1987 LANDUSE

NB = 54 NP = 32 IFIN = 250
50. YR AND 6. HR DURATION STORM = 2.90 IN
TIME STEP = 30. MINUTES

T/TP = 0.00 .10 .20 .30 .40 .50 .60 .70 .80 .90 1.00 1.10 1.20 1.30
1.40 1.50 1.60 1.70 1.80 1.90 2.00 2.20 2.40 2.60 2.80 3.00 3.20 3.40
3.60 4.00 4.50 5.00
Q/QP = 0.00 .03 .10 .19 .31 .47 .66 .82 .93 .99 1.00 .99 .93 .86
.78 .68 .56 .46 .39 .33 .28 .21 .15 .11 .08 .06 .04 .03
.02 .01 .01 0.00

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	21.59	4.26	70.00	358.83
NC2	8.67	4.77	13.08	5.86	70.00	728.09
NC3	7.00	5.87	18.45	6.52	70.00	516.01
NC4	4.44	6.95	19.40	7.17	70.00	287.50
NC5	8.02	8.35	17.46	8.01	70.00	450.98
NC6	3.63	3.06	16.27	4.84	70.00	379.15
NC7	6.33	6.07	16.27	6.64	70.00	456.25
NC8	26.00	19.65	12.50	14.79	70.00	681.34
NC9	8.39	15.87	13.29	12.52	70.00	269.07
MC1	10.31	7.03	23.24	7.22	70.00	659.32
MC2	10.83	16.30	20.62	12.78	70.00	338.39
MC3	9.33	10.79	17.66	9.47	70.00	425.06
MC4	2.59	1.33	18.25	3.80	70.00	306.01
MC5	.75	2.22	17.66	4.33	70.00	88.27
MC6	5.06	10.95	18.81	9.57	70.00	228.82
MC7	7.22	7.14	16.47	7.28	70.00	455.80
MC8	10.65	9.95	13.49	8.97	70.00	521.58
SC1	4.40	9.72	23.59	8.83	70.00	219.80
SC2	5.38	14.90	20.57	11.94	70.00	200.52
SC3	3.12	12.73	25.47	10.64	70.00	122.02
SC4	2.53	6.09	22.46	6.65	70.00	181.79
SC5	2.84	4.97	18.91	5.98	70.00	235.71
SC6	1.43	3.87	17.49	5.32	70.00	133.07
SC7	3.09	9.26	19.86	8.56	70.00	159.24
SC8	4.90	4.10	18.91	5.46	70.00	452.32
SC9	2.09	4.62	17.49	5.77	70.00	177.46
SC10	3.71	4.93	16.08	5.96	70.00	306.17
C1	.90	.62	19.74	3.37	70.00	111.70
C2	4.70	2.37	18.68	4.42	70.00	525.12
C3	6.05	13.40	14.89	11.04	70.00	226.51
C4	3.75	7.92	14.89	7.75	70.00	221.75
C5	7.94	16.66	13.48	13.00	70.00	242.86
C6	12.95	23.34	11.47	17.30	70.00	291.95
C7	8.73	29.54	10.76	20.72	70.00	154.16
C8	6.90	14.87	11.47	11.92	70.00	235.76
C9	5.04	14.62	11.47	11.77	70.00	174.20
C10	3.28	12.40	8.51	10.44	70.00	131.75
C11	5.50	13.37	8.51	11.02	70.00	206.36
C12	10.89	29.24	8.51	20.54	70.00	194.25
C13	4.25	11.11	9.46	9.67	70.00	188.56
C14	9.51	28.59	9.46	20.15	70.00	173.43
C15	10.28	17.41	6.62	13.45	71.55	332.83
C16	7.57	11.53	6.62	9.92	72.28	372.45
C17	10.83	20.83	6.62	15.50	72.39	312.00
C18	8.73	10.19	6.38	9.11	71.09	450.92
C19	2.00	7.44	5.44	7.46	76.65	184.07
C20	.74	7.44	5.44	7.46	81.84	87.89
C21A	2.97	9.66	7.40	8.80	74.25	191.93
C21B	5.07	11.11	5.41	9.67	74.87	300.95
C21C	5.91	10.45	3.55	9.27	74.01	351.96
C22	4.76	16.50	4.02	12.90	79.20	249.65
C23	9.58	11.93	4.02	10.16	72.61	468.80
C24	7.66	17.22	0.00	13.33	70.13	229.78
C25	5.82	10.75	0.00	9.45	70.83	280.43

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	0.	1.	5.	11.	22.	37.	59.	88.
126.	174.	235.	310.	402.	515.	654.	817.	1010.	1230.	1469.	1721.
1989.	2262.	2519.	2760.	2976.	3164.	3323.	3473.	3599.	3722.	3856.	4014.
4185.	4376.	4605.	4863.	5144.	5493.	5894.	6356.	6916.	7504.	8035.	8480.
8826.	9029.	9072.	9000.	8757.	8463.	8143.	7813.	7424.	6950.	6476.	6075.
5759.	5483.	5225.	4966.	4692.	4414.	4136.	3853.	3577.	3312.	3058.	2821.
2598.	2391.	2198.	2027.	1869.	1721.	1584.	1457.	1337.	1226.	1124.	1030.
945.	868.	800.	736.	678.	625.	577.	533.	493.	456.	423.	392.
363.	336.	311.	289.	269.	251.	233.	218.	203.	190.	178.	166.
156.	146.	137.	128.	120.	112.	105.	98.	92.	85.	80.	74.
69.	65.	61.	57.	53.	50.	47.	44.	42.	39.	36.	34.
32.	30.	28.	26.	24.	23.	21.	20.	19.	18.	17.	16.
15.	14.	13.	13.	12.	11.	10.	10.	9.	9.	8.	7.
6.	6.	6.	5.	5.	4.	4.	4.	4.	3.	3.	3.
3.	3.	3.	2.	2.	2.	2.	2.	2.	1.	1.	1.
1.	1.	1.	1.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.			

CROW CREEK (INCLUDING CHEYENNE) FLOOD HYDROGRAPH - FOR 2 YR 24 HR STORM **TABLE B.4-10**
1987 LANDUSE, WITH PROJECT IMPACT

NB = 54 NP = 32 IFIN = 250
2. YR AND 24. HR DURATION STORM = 1.68 IN
TIME STEP = 30. MINUTES

T/TP = 0.00 .10 .20 .30 .40 .50 .60 .70 .80 .90 1.00 1.10 1.20 1.30
1.40 1.50 1.60 1.70 1.80 1.90 2.00 2.20 2.40 2.60 2.80 3.00 3.20 3.40
3.60 4.00 4.50 5.00
Q/QP = 0.00 .03 .10 .19 .31 .47 .66 .82 .93 .99 1.00 .99 .93 .86
.78 .68 .56 .46 .39 .33 .28 .21 .15 .11 .08 .06 .04 .03
.02 .01 .01 0.00

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	21.59	13.26	70.00	60.41
NC2	8.67	4.77	13.08	14.86	70.00	67.46
NC3	7.00	5.87	18.45	15.52	70.00	53.53
NC4	4.44	6.95	19.40	16.17	70.00	32.78
NC5	8.02	8.35	17.46	17.01	70.00	55.29
NC6	3.63	3.06	16.27	13.84	70.00	29.48
NC7	6.33	6.07	16.27	15.64	70.00	48.20
NC8	26.00	19.65	12.50	23.79	70.00	121.07
NC9	8.39	15.87	13.29	21.52	70.00	45.51
MC1	10.31	7.03	23.24	16.22	70.00	75.66
MC2	10.83	16.30	20.62	21.78	70.00	57.23
MC3	9.33	10.79	17.66	18.47	70.00	60.53
MC4	2.59	1.33	18.25	12.80	70.00	20.75
MC5	.75	2.22	17.66	13.33	70.00	6.26
MC6	5.06	10.95	18.81	18.57	70.00	32.92
MC7	7.22	7.14	16.47	16.28	70.00	52.89
MC8	10.65	9.95	13.49	17.97	70.00	71.94
SC1	4.40	9.72	23.59	17.83	70.00	29.95
SC2	5.88	14.90	20.57	20.94	70.00	32.70
SC3	3.12	12.73	25.47	19.64	70.00	18.93
SC4	2.53	6.09	22.46	15.65	70.00	19.20
SC5	2.84	4.97	18.91	14.98	70.00	22.21
SC6	1.43	3.87	17.49	14.32	70.00	11.22
SC7	3.09	9.26	19.86	17.56	70.00	21.42
SC8	4.90	4.10	18.91	14.46	70.00	38.80
SC9	2.09	4.62	17.49	14.77	70.00	16.36
SC10	3.71	4.93	16.08	14.96	70.00	28.82
C1	.90	.62	19.74	12.37	70.00	7.40
C2	4.70	2.37	18.68	13.42	70.00	38.05
C3	6.05	13.40	14.39	20.04	70.00	35.88
C4	3.75	7.92	14.89	16.75	70.00	26.95
C5	7.94	16.66	13.48	22.00	70.00	41.07
C6	12.95	23.84	11.47	26.30	70.00	51.98
C7	8.73	29.54	10.76	29.72	70.00	29.28
C8	6.90	14.87	11.47	20.92	70.00	38.45
C9	5.04	14.62	11.47	20.77	70.00	28.40
C10	3.28	12.40	8.51	19.44	70.00	20.11
C11	5.50	13.37	8.51	20.02	70.00	32.69
C12	10.89	29.24	8.51	29.54	70.00	36.94
C13	4.25	11.11	9.46	18.67	70.00	27.39
C14	9.51	28.59	9.46	29.15	70.00	32.87
C15	10.28	17.41	6.62	22.45	71.56	62.60
C16	7.57	11.53	6.62	18.92	72.30	61.24
C17	10.83	20.83	6.62	24.50	72.42	64.32
C18	8.73	10.19	6.38	18.11	71.10	65.80
C19	2.00	7.44	5.44	16.46	76.71	27.25
C20	.74	7.44	5.44	16.46	81.97	15.21
C21A	2.97	9.66	7.40	17.80	74.42	30.87
C21B	5.07	11.11	5.41	18.67	75.12	54.24
C21C	5.91	10.45	3.55	18.27	74.05	58.58
C22	4.76	16.50	4.02	21.90	79.39	61.62
C23	9.58	11.93	4.02	19.16	72.63	79.82
C24	7.66	17.22	0.00	22.33	70.13	39.92
C25	5.82	10.75	0.00	18.45	70.83	41.52

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	1.	1.	2.	4.	6.	10.	15.	21.	29.	40.
53.	68.	86.	107.	130.	156.	184.	214.	245.	277.	310.	343.
377.	411.	444.	478.	513.	548.	583.	620.	657.	695.	734.	773.
815.	861.	910.	962.	1016.	1070.	1123.	1173.	1221.	1267.	1311.	1352.
1390.	1422.	1447.	1468.	1485.	1499.	1505.	1506.	1501.	1490.	1472.	1445.
1412.	1371.	1323.	1269.	1206.	1141.	1080.	1023.	965.	905.	845.	790.
739.	693.	648.	605.	564.	524.	486.	449.	415.	382.	352.	325.
299.	275.	253.	233.	214.	197.	181.	167.	154.	141.	130.	120.
111.	102.	94.	87.	81.	75.	69.	64.	60.	55.	52.	48.
45.	42.	39.	36.	34.	31.	29.	27.	26.	24.	22.	21.
20.	18.	17.	16.	15.	14.	13.	12.	12.	11.	10.	9.
9.	8.	8.	7.	7.	6.	6.	6.	5.	5.	5.	4.
4.	4.	4.	3.	3.	3.	3.	3.	2.	2.	2.	2.
2.	2.	2.	2.	1.	1.	1.	1.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.		

CROW CREEK (INCLUDING CHEYENNE) FLOOD HYDROGRAPH - FOR 10 YR 3 HR STORM
1987 LANDUSE, WITH PROJECT IMPACT

TABLE B.4-11

NB = 54 NP = 32 IFIN = 250
10. YR AND 3. HR DURATION STORM = 1.71 IN
TIME STEP = 30. MINUTES

T/TP = 0.00 .10 .20 .30 .40 .50 .60 .70 .80 .90 1.00 1.10 1.20 1.30
1.40 1.50 1.60 1.70 1.80 1.90 2.00 2.20 2.40 2.60 2.80 3.00 3.20 3.40
3.60 4.00 4.50 5.00

Q/QP = 0.00 .03 .10 .19 .31 .47 .66 .82 .93 .99 1.00 .99 .93 .86
.78 .68 .56 .46 .39 .33 .28 .21 .15 .11 .08 .06 .04 .03
.02 .01 .01 0.00

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	21.39	2.76	70.00	296.48
NC2	8.67	4.77	13.08	4.36	70.00	188.28
NC3	7.00	5.87	18.45	5.02	70.00	125.38
NC4	4.44	6.95	19.40	5.67	70.00	67.91
NC5	8.02	8.35	17.46	6.51	70.00	104.34
NC6	3.63	3.06	16.27	3.34	70.00	114.47
NC7	6.33	6.07	16.27	5.14	70.00	109.49
NC8	26.00	19.65	12.50	13.29	70.00	148.37
NC9	8.39	15.87	13.29	11.02	70.00	58.96
MC1	10.31	7.03	23.24	5.72	70.00	155.34
MC2	10.83	16.30	20.62	11.28	70.00	74.15
MC3	9.33	10.79	17.66	7.97	70.00	95.15
MC4	2.59	1.33	18.25	2.30	70.00	124.21
MC5	.75	2.22	17.66	2.83	70.00	29.22
MC6	5.06	10.95	18.81	8.07	70.00	50.89
MC7	7.22	7.14	16.47	5.78	70.00	107.62
MC8	10.65	9.95	13.49	7.47	70.00	117.41
SC1	4.40	9.72	23.59	7.33	70.00	49.62
SC2	5.88	14.90	20.57	10.44	70.00	43.94
SC3	3.12	12.73	25.47	9.14	70.00	27.13
SC4	2.53	6.09	22.46	5.15	70.00	43.63
SC5	2.84	4.97	18.91	4.48	70.00	58.60
SC6	1.43	3.87	17.49	3.82	70.00	36.60
SC7	3.09	9.26	19.86	7.06	70.00	36.42
SC8	4.90	4.10	18.91	3.96	70.00	119.11
SC9	2.09	4.62	17.49	4.27	70.00	46.71
SC10	3.71	4.93	16.08	4.46	70.00	77.12
C1	.90	.62	19.74	1.87	70.00	60.45
C2	4.70	2.37	18.68	2.92	70.00	174.58
C3	6.05	13.40	14.89	9.54	70.00	50.06
C4	3.75	7.92	14.89	6.25	70.00	51.30
C5	7.94	16.66	13.48	11.50	70.00	53.22
C6	12.95	23.84	11.47	15.80	70.00	61.17
C7	8.73	29.54	10.76	19.22	70.00	33.40
C8	6.90	14.87	11.47	10.42	70.00	51.66
C9	5.04	14.62	11.47	10.27	70.00	38.36
C10	3.28	12.40	8.51	8.94	70.00	29.25
C11	5.50	13.37	8.51	9.52	70.00	45.61
C12	10.89	29.24	8.51	19.04	70.00	42.09
C13	4.25	11.11	9.46	8.17	70.00	42.06
C14	9.51	28.59	9.46	18.65	70.00	37.58
C15	10.28	17.41	6.62	11.95	71.56	79.32
C16	7.57	11.53	6.62	8.42	72.30	95.31
C17	10.83	20.33	6.62	14.00	72.42	77.90
C18	8.73	10.19	6.38	7.61	71.10	107.50
C19	2.00	7.44	5.44	5.96	76.71	59.80
C20	.74	7.44	5.44	5.96	81.97	34.74
C21A	2.97	9.66	7.40	7.30	74.42	55.69
C21B	5.07	11.11	5.41	8.17	75.12	88.91
C21C	5.91	10.45	3.55	7.77	74.05	98.90
C22	4.76	16.50	4.02	11.40	79.39	84.93
C23	9.58	11.93	4.02	8.66	72.63	121.05
C24	7.66	17.22	0.00	11.83	70.13	50.57
C25	5.92	10.75	0.00	7.95	70.83	66.00

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	1.	4.	9.	16.	23.	31.	42.	57.
75.	98.	132.	173.	220.	274.	342.	423.	504.	580.	650.	716.
780.	831.	868.	897.	919.	936.	949.	967.	998.	1044.	1093.	1143.
1175.	1214.	1236.	1376.	1483.	1637.	1808.	1962.	2065.	2100.	2023.	1940.
1913.	1934.	1912.	1793.	1632.	1489.	1390.	1320.	1267.	1216.	1159.	1102.
1044.	980.	914.	850.	789.	732.	675.	620.	567.	522.	482.	444.
410.	379.	349.	321.	295.	268.	245.	225.	207.	191.	175.	162.
149.	137.	126.	116.	108.	101.	93.	86.	79.	73.	68.	64.
59.	55.	51.	48.	45.	42.	39.	36.	34.	32.	30.	28.
26.	25.	23.	21.	20.	19.	17.	16.	15.	14.	13.	12.
12.	11.	10.	10.	9.	9.	8.	7.	7.	7.	6.	6.
5.	5.	5.	4.	4.	4.	4.	3.	3.	3.	3.	3.
3.	2.	2.	2.	2.	2.	2.	2.	1.	1.	1.	1.
1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	1.	1.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.		

CROW CREEK (INCLUDING CHEYENNE) FLOOD HYDROGRAPH - FOR 50 YR 6 HR STORM
1987 LANDUSE, WITH PROJECT IMPACT

TABLE B.4-12

NB = 54 NP = 32 IFIN = 250
50. YR AND 6. HR DURATION STORM = 2.90 IN
TIME STEP = 30. MINUTES

T/TP =	0.00	.10	.20	.30	.40	.50	.60	.70	.80	.90	1.00	1.10	1.20	1.30
	1.40	1.50	1.60	1.70	1.80	1.90	2.00	2.20	2.40	2.60	2.80	3.00	3.20	3.40
	3.60	4.00	4.50	5.00										
Q/QP =	0.00	.03	.10	.19	.31	.47	.66	.82	.93	.99	1.00	.99	.93	.86
	.78	.68	.56	.46	.39	.33	.28	.21	.15	.11	.08	.06	.04	.03
	.02	.01	.01	0.00										

BASIN	AREA(SQ.M)	TC(HR)	TT(HR)	TP(HR)	CN	QP
NC1	7.40	2.10	21.59	4.26	70.00	858.83
NC2	8.67	4.77	13.08	5.86	70.00	728.09
NC3	7.00	5.87	18.45	6.52	70.00	516.01
NC4	4.44	6.95	19.40	7.17	70.00	287.50
NC5	8.02	8.35	17.46	9.01	70.00	450.98
NC6	3.63	3.06	16.27	4.84	70.00	379.15
NC7	6.33	6.07	16.27	6.64	70.00	456.25
NC8	26.00	19.65	12.50	14.79	70.00	681.34
NC9	8.39	15.87	13.29	12.52	70.00	269.07
MC1	10.31	7.03	23.24	7.22	70.00	659.32
MC2	10.83	16.30	20.62	12.78	70.00	338.39
MC3	9.33	10.79	17.66	9.47	70.00	425.06
MC4	2.59	1.33	18.25	3.80	70.00	306.01
MC5	.75	2.22	17.66	4.33	70.00	88.27
MC6	5.06	10.95	18.81	9.57	70.00	228.32
MC7	7.22	7.14	16.47	7.28	70.00	455.80
MC8	10.65	9.95	13.49	8.97	70.00	521.58
SC1	4.40	9.72	23.59	8.83	70.00	219.80
SC2	5.88	14.90	20.57	11.94	70.00	200.52
SC3	3.12	12.73	25.47	10.64	70.00	122.02
SC4	2.53	6.09	22.46	6.65	70.00	181.79
SC5	2.84	4.97	18.91	5.98	70.00	235.71
SC6	1.43	3.87	17.49	5.32	70.00	133.07
SC7	3.09	9.26	19.86	8.56	70.00	159.24
SC8	4.90	4.10	18.91	5.46	70.00	452.32
SC9	2.09	4.62	17.49	5.77	70.00	177.46
SC10	3.71	4.93	16.08	5.96	70.00	306.17
C1	.90	.62	19.74	3.37	70.00	111.70
C2	4.70	2.37	18.68	4.42	70.00	525.12
C3	6.05	13.40	14.89	11.04	70.00	226.51
C4	3.75	7.92	14.89	7.75	70.00	221.75
C5	7.94	16.66	13.48	13.00	70.00	242.86
C6	12.95	23.84	11.47	17.30	70.00	281.95
C7	8.73	29.54	10.76	20.72	70.00	154.16
C8	6.90	14.87	11.47	11.92	70.00	235.76
C9	5.04	14.62	11.47	11.77	70.00	174.20
C10	3.28	12.40	8.51	10.44	70.00	131.75
C11	5.50	13.37	8.51	11.02	70.00	206.36
C12	10.89	29.24	8.51	20.54	70.00	194.25
C13	4.25	11.11	9.46	9.67	70.00	188.56
C14	9.51	28.59	9.46	20.15	70.00	173.43
C15	10.28	17.41	6.62	13.45	71.56	333.04
C16	7.57	11.53	6.62	9.92	72.30	372.90
C17	10.83	20.83	6.62	15.50	72.42	312.57
C18	8.73	10.19	6.38	9.11	71.10	451.20
C19	2.00	7.44	5.44	7.46	76.71	184.65
C20	.74	7.44	5.44	7.46	81.97	88.42
C21A	2.97	9.66	7.40	8.80	74.42	193.80
C21B	5.07	11.11	5.41	9.67	75.12	305.16
C21C	5.91	10.45	3.55	9.27	74.05	352.77
C22	4.76	16.50	4.02	12.90	79.39	252.12
C23	9.58	11.93	4.02	10.16	72.63	469.36
C24	7.66	17.22	0.00	13.33	70.13	229.98
C25	5.82	10.75	0.00	9.45	70.83	280.43

FLOOD HYDROGRAPH IN EVERY 30. MINUTES

0.	0.	0.	0.	0.	1.	5.	11.	22.	37.	59.	88.
126.	174.	235.	311.	403.	517.	656.	820.	1014.	1235.	1475.	1723.
1996.	2270.	2529.	2770.	2987.	3175.	3345.	3485.	3610.	3733.	3867.	4024.
4195.	4385.	4613.	4871.	5151.	5499.	5899.	6362.	6921.	7508.	8039.	8484.
8829.	9032.	9074.	9002.	8760.	8465.	8145.	7814.	7426.	6951.	6478.	6076.
5760.	5484.	5226.	4967.	4693.	4414.	4137.	3854.	3577.	3312.	3058.	2822.
2599.	2391.	2199.	2027.	1869.	1721.	1585.	1457.	1337.	1226.	1124.	1030.
945.	868.	800.	736.	678.	625.	577.	533.	493.	456.	423.	392.
363.	336.	311.	289.	269.	251.	233.	218.	203.	190.	178.	166.
156.	146.	137.	129.	120.	112.	105.	98.	92.	85.	80.	74.
69.	65.	61.	57.	53.	50.	47.	44.	42.	39.	36.	34.
32.	30.	28.	26.	24.	23.	21.	20.	19.	18.	17.	16.
15.	14.	13.	13.	12.	11.	10.	10.	9.	8.	8.	7.
6.	6.	6.	5.	5.	4.	4.	4.	4.	3.	3.	3.
3.	3.	3.	2.	2.	2.	2.	2.	2.	1.	1.	1.
1.	1.	1.	1.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
0.	0.	0.	0.	0.	0.	0.	0.	0.	0.		